A STUDY OF THE BEHAVIOR OF REINFORCED CONCRETE BEAMS SUBJECTED TO REPEATED LOADS

NOVEMBER 1967

NO.30

by

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Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

A STUDY OF THE BEHAVIOR OF REINFORCED
CONCRETE BEAMS SUBJECTED TO REPEATED LOADS

To: G. A. Leonards, Director
Joint Highway Research Project

From: H. L. Michael, Associate Director
Joint Highway Research Project

December 28, 1967

File: 7-4-13

Project: C-36-65M

The attached Final Report "A Study of the Behavior of Reinforced Concrete Beams Subjected to Repeated Loads" has been authored by Mr. William A. Rogers, Graduate Assistant in the Structural Engineering area. Mr. Rogers also used the research report as his thesis in partial fulfillment of the requirements for the MSCE degree. Professors M. J. Gutzwiller and R. H. Lee directed the research project.

The objective of the study was to observe the behavior of reinforced concrete beams of different shear span-to-span depth ratio with varying amounts of web reinforcement under repeated loads. The report contains a detailed discussion of the failure patterns and individual beam behavior as well as a summary of test results.

The report is presented for the record as fulfillment of the objectives of this research as approved by the Advisory Board on March 22, 1966.

Respectfully submitted,

Harold L. Michael/jgs

Harold L. Michael
Associate Director

HLM:nf

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Final Report

A STUDY OF THE BEHAVIOR OF REINFORCED CONCRETE BEAMS SUBJECTED TO REPEATED LOADS

by

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Joint Highway Research Project
File: 7-4-13
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December 28, 1967

ACKNOWLEDGMENTS

Acknowledgment is made to the members of the board of The Joint Highway Research Project and to Professor G. A. Leonards, Director, for providing funds for the project.

Special thanks are given to Professors Martin J. Gutzwiller and Robert H. Lee, major professors, and to Professor E. O. Stitz for their suggestions and guidance.

The author also wishes to express his appreciation to Mr. G. W. Foster and Mr. W. E. Telfer, laboratory technicians, and to the several graduate students for their generous assistance in the laboratory.

Special thanks are also given to the author's wife and family for their assistance and encouragement.

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LIST OF SYMBOLS

A_s	nominal area of tension steel
A'_s	nominal area of compression steel
A_v	cross-sectional area of one stirrup
a	length of critical shear span (distance from section of maximum moment to point of inflection)
b	width of beam section
C'	total internal compression force in concrete
d	effective depth (measured to centroid of tension steel)
d'	distance from compression face to centroid of compression steel
jd	internal moment arm, straight-line theory
E_s	modulus of elasticity of steel
E_c	initial tangent modulus of concrete
f'_c	concrete compressive strength, 6" x 12" standard cylinder
f_v	stress in stirrup
f_{vy}	yield strength of stirrup steel

LIST OF SYMBOLS (continued)

f_s	stress in longitudinal tension steel
f'_s	stress in longitudinal compression steel
f_{su}	stress in tension steel at failure of beam
f_y	yield strength of longitudinal steel
M	moment at any section
M_s	ultimate shear-compression moment
M_u	ultimate flexural moment
M_1	maximum negative moment
M_2	maximum positive moment
n	E_s/E_c modular ratio
N	number of cycles of loading
p	A_s/bd percentage longitudinal tension steel, total load
P_1, P_2	load on the overhang and load between supports, respectively
P_c	total load at formation of diagonal tension crack
P_f	predicted ultimate load
P_u	total load at failure
r	A_v/bs - web reinforcement ratio
s	horizontal spacing of stirrups
T	total force in tension steel

LIST OF SYMBOLS (continued)

V	total shear at any section
V_a	shear in critical shear span
V_c	shear assigned to concrete (working stress design)
V'	shear assigned to stirrups (working stress design)
v	nominal shearing stress = v/bjd or v/bd as defined in text
v_c	portion of total shearing stress assigned to concrete or average shear at diagonal cracking
v_s	portion of total shearing stress assigned to stirrups
v_a	allowable nominal shearing stress (v/bjd)
v_u	ultimate shear strength
α	inclination of stirrups with respect to longitudinal axis
θ	inclination of diagonal crack with respect to longitudinal axis
S.C.	shear-compression failure
D.T.	diagonal tension failure
F.R.	fatigue of reinforcement failure
MII	strain in micro-inches per inch

ABSTRACT

Rogers, William A., MSCE, Purdue University, January, 1968. A STUDY OF THE BEHAVIOR OF REINFORCED CONCRETE BEAMS SUBJECTED TO REPEATED LOADS. Major Professors: M. J. Gutzwiller and R. H. Lee.

Sixteen beams of 6" x 13" rectangular cross-section were subjected to repeated loading in such a manner as to simulate a portion of a continuous girder subjected to concentrated loads. The beams were designed so that the critical region for failure with respect to shear was the length between the point of zero moment and the point of maximum negative moment -- commonly called the shear span.

The objective of this study was to observe the behavior of beams of different shear span-to-depth ratio with varying amounts of web reinforcement. The magnitude of the repeated load was taken as a percentage of the predicted ultimate load and was varied to determine the effect on the behavior of the specimens.

The specimens, which were weak with respect to shear, failed in one of three modes: shear-compression, diagonal tension, or brittle fracture of the reinforcement. It was found that the fatigue life of the member increased when the magnitude of the repeated load was reduced. The presence of stirrups was observed to increase the endurance of a member

when compared to the behavior of a similar specimen without web reinforcement.

Detailed discussion of the failure patterns and individual beam behavior are presented along with the summary of test results.

INTRODUCTION

Reinforced concrete has been, and continues to be, the subject of extensive experimental and analytical research. The nonhomogeneous nature of concrete requires extensive experimental corroboration of analytical studies of the behavior of concrete members subjected to load. The results of previous investigations have provided a reasonable understanding of the modes of failure and ultimate strength of reinforced concrete members subjected to pure flexure, combined flexure and axial load, and axial compression.

There have been numerous investigations to determine the strength of reinforced concrete members under nearly all types of loading situations. However, a majority of the testing of reinforced concrete members has been done with the use of the laboratory testing machine for short-time loading or with sustained loading. Both of these types of loading can be characterized as monotonically increasing in nature and represent the case of static loading. It is upon these loading situations that current design practice is predicted with appropriate factors of uncertainty included for insurance against failure.

Investigations using repeated loading have been conducted since the early 1900's to examine the behavior of plain and

reinforced concrete when subjected to fatigue (repeated) loadings. However, a review of the literature reporting the experimental work with repeated loadings suggests that there is much to be learned if engineers are to have a reasonably reliable basis for executing economical designs.

As the methods of design and analysis of reinforced concrete structures become more precise for reasons of economy and safety, the need for more information concerning the performance of reinforced concrete members under various loading criteria has increased significance.

The behavior of reinforced concrete members that are weak with respect to shear has been the subject of intensive investigation in the past decade. However, most of the research work has been conducted using static loading. This fact suggests that a study of reinforced concrete members which are weak in shear and are subjected to repetitive loads would be useful addition to the knowledge of reinforced concrete.

In order to define the conditions under which the strength of a reinforced concrete member will be governed by shear, one might consider the case of a simply-supported beam that is subjected to two equal loads which are symmetrically placed with respect to the center of the span. At the present it will be assumed that these loads are static. When the loads are positioned near the center of the span, the beam has a large shear span to depth ratio (i.e., the length of the span from the reaction to the load, which is referred to

as the shear span, is great), and the effect of shear on the ultimate strength of the beam is negligibly small. The failure mode for this situation will be that of flexure with either the crushing of the concrete or the initial yielding of the tensile reinforcement followed by crushing of the concrete. This condition of loading approximates the case of pure flexure for which an accurate prediction of the ultimate load can be made.

When the two equal loads are moved toward the supports, the shear span to depth ratio is decreased, and the influence of shear becomes apparent. At the first application of load flexural cracks will form as in the case of pure flexure. However, as the load is increased, the flexural cracks will begin to incline toward the load. This observed inclination of the typically vertical flexural cracks can be explained by the combination of shearing stresses and tensile bending stresses which produce a principal tension acting at an inclination of approximately 45° near the neutral axis and nearly horizontal at the bottom of the beam. There may also be an independent formation of diagonal tension cracks near the neutral axis. Once the diagonal tension crack has formed, there are two typical modes of behavior which have been observed. One mode is that the diagonal tension crack, once formed, will propagate immediately, or with a slight increase in load, cross the entire cross-section from the tensile reinforcement to the compression face, split it into two separate pieces and thus fail the beam. This type of failure

is referred to as a diagonal tension failure and occurs suddenly and without warning, and is most frequently observed in beams with moderate shear span to depth ratios (i.e., ratios from approximately 3.4 to 6). The alternate mode is that the diagonal tension crack, once formed, will propagate and partially penetrate the compression zone. The beam will then fail ultimately by crushing of the concrete. This type of failure is referred to as a shear compression failure, and is usually observed in beams with smaller shear span to depth ratios (i.e., ratios less than approximately 3.4). There is no sudden collapse as in the case of a diagonal tension failure, and the ultimate load is usually significantly higher than the load at which the diagonal tension crack first occurs.

One may conclude in a qualitative manner that shear affects the behavior of beams through the formation of diagonal tension cracks. In some instances, the formation of a diagonal tension crack may result in concurrent failure of the beam. However, there are situations where shear is not an important consideration since a beam may fail before the load level ever causes the formation of a diagonal tension crack. Therefore, shear will not affect the ultimate strength of a member if a diagonal tension crack does not form before the ultimate load is reached.

The formation of a critical diagonal tension crack is accompanied by an internal redistribution of stresses within a beam. Shear stresses cannot be transferred along the crack;

therefore, the longitudinal reinforcement must transfer some of the shear force by dowel action while the uncracked concrete cross-section must resist the remainder. Until the formation of the diagonal crack, the stress in the longitudinal steel and in the concrete is proportional to the moment in the beam.

Figure 1a shows a beam without web reinforcement in which a diagonal tension crack has formed. The shear and moment diagrams are shown in Figure 1b. The free body of a portion of the beam outside of the crack is shown in Figure 1c. After the formation of the diagonal tension crack, the area of the concrete above the crack is assumed to resist the entire shear force at the weakened section. This assumption is reasonably valid as the transfer of the shear force by dowel action of the longitudinal reinforcement is small. The remaining forces acting on the free body are the tensile force in the steel (_T_), the compression force on the concrete (_C'_), and the reaction (_R_).

When the forces in Figure 1c are in equilibrium, the summation of moments about the centroid of compression in the uncracked portion of the beam (section _b-b_) shows that the steel at section _a-a_ must carry an increased tension because of the diagonal crack. At first there is probably some dowel action at section _a-a_, but as the crack increases in width and rotation tends to concentrate about the reduced section, the dowel forces are greatly increased and soon destroyed through additional cracking along the steel. The

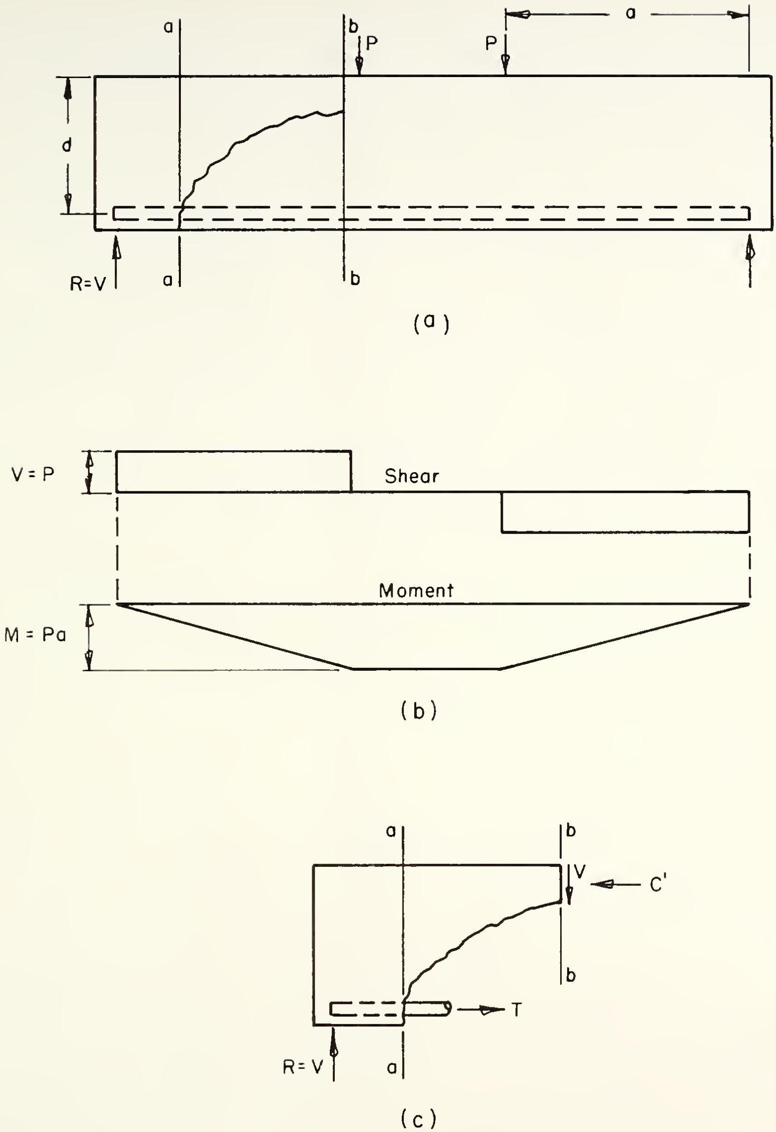


FIGURE I. REDISTRIBUTION OF INTERNAL STRESSES

ability of a beam to reach a force equilibrium after the redistribution of forces seems to depend primarily on the stability of the compression zone, but it is also influenced by the length of the shear span and the percentage of longitudinal reinforcement.

Many specimens tested in the course of shear investigations have failed in the shear-compression mode at loads which were twice the load that caused the original diagonal crack. However, there were a number of specimens that failed simultaneously with the formation of the diagonal tension crack. As a result, the diagonal cracking load is taken as the ultimate load for a beam without web reinforcement. Web reinforcement has little or no effect on the behavior of a beam prior to diagonal cracking. Experimental measurements have shown that the stress in the stirrups is negligible before the formation of the diagonal crack. Once the diagonal crack has formed, the stirrups which are crossed by the crack help resist the shear force originally carried by the uncracked concrete cross section. With increases in the load, the stirrups crossed by the crack are stressed in proportion to the additional load.

Two benefits of shear reinforcement in addition to carrying the load are: (1) The stirrups help restrict the growth of diagonal cracks and reduce penetration of the cracks into the compression zone. (2) The stirrups provide restraint against splitting along the longitudinal steel by tying the reinforcement to the mass of the concrete beam.

With the use of shear reinforcement the undesirable sudden failure which is characteristic of diagonal tension failures can usually be converted to the flexure mode or to a mode associated with the yield of the stirrups. Because of the erratic behavior of the diagonal crack after it has formed, purely analytical predictions are not reliable. However, present design concepts have evolved through a combination of rational analysis, test evidence and experience.

Development of Design Procedures

The problem of designing reinforced concrete members to resist shear forces arises from the fact that concrete has a relatively low tensile strength in comparison to its compressive strength. Consider a simple beam subjected to loads. The only stresses acting at the neutral axis are shear stresses. However, these shear stresses can be resolved into two pairs of principal stresses; one of the pairs is compressive while the other is tensile. At other depths in the beam there may be either normal compressive or tensile stresses acting together with shear stresses. In these areas there is a problem of combined stresses acting, which produces a situation of principal stresses that are tensile and compressive in nature. Thus, for a simple flexural member there exist trajectories along which the principal stresses are tensile and compressive. These tensile stresses, which exist in all parts of a beam, are known as diagonal tension stresses, and may be critical at other depths of the beam rather than at the extreme surface.

In order to develop a criterion for design early investigators used as a measure of the diagonal tension a nominal unit shear stress derived on the basis that the concrete below the neutral axis carries no tension. This criterion was originally suggested by E. Mörsch of Germany(10)*, and recognizes that the problem of shear in reinforced concrete members is a problem of combined stresses. The expression proposed by Mörsch is derived as follows. Figure 2a illustrates a simple beam which is subjected to concentrated loads and has constant shear. Consider an infinitesimal length Δx of the beam, subjected to a constant shear $\underline{V}$ and moments $\underline{M}$ and $\underline{M + \Delta M}$. Summation of moments about point A in Figure 2b must be equal to zero; therefore

$$\Delta T \; jd = V \Delta x$$

or

$$\Delta T = \frac{V \Delta x}{jd}$$

Summation of horizontal forces in Figure 2c must also be equal to zero; therefore

$$\Delta T = vb(\Delta x)$$

Equating these two values for ΔT gives:

$$v = \frac{V}{bjd} \qquad (1)$$

* Superscripted numbers in parentheses refer to items in the Bibliography.

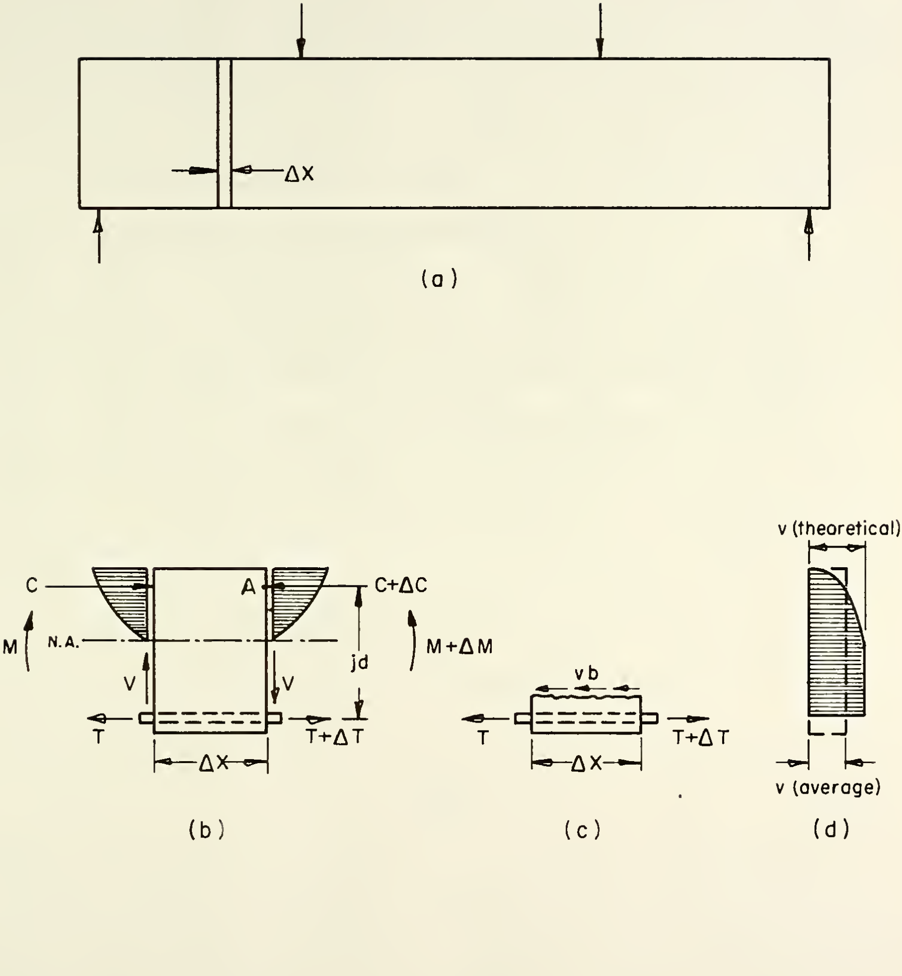


FIGURE 2. SHEAR STRESSES

The derivation assumes that the unit shear stress is distributed parabolically in the compression zone and is constant below the neutral axis as shown in Figure 2d. This is only a nominal or average shear stress, based on the assumption that concrete carries no tensile bending stress. The actual distribution of these shear stresses over the cross-section is unknown. This derivation ignores the influence of the reinforcement, the inelastic behavior of concrete, and the shear concentrations at moment cracks.

Mörsch's expression has been used almost universally as a measure of diagonal tension for a basis to design for shear. An allowable value for this nominal shear stress is then specified. Most American design codes have designated the allowable shearing stress as a percentage of the concrete cylinder strength with a maximum upper limit for members without web reinforcement. The current ASSHO specifications[4] (1965) allow a nominal unit shear of 0.03 f'_c with a maximum of 90 psi. The 1963 ACI Building Code[3] redefined the nominal shear stress as:

$$v = \frac{V}{bd} \tag{2}$$

This action was in accordance with recommendations made by a joint ASCE-ACI Committee 326[2] on shear and diagonal tension. The committee felt that the load which produced the first diagonal tension crack should be taken as the limiting value for beams without web reinforcement. They recognized that the load capacity of a reinforced concrete member was a

function of the concrete cylinder strength (f'_c), the moment to shear ratio (M/Vd), and the percentage of longitudinal steel (p); but it was realized that the relationship would have to be empirical based on the data from laboratory tests. Thus, for the case of beams without web reinforcement the expression for the cracking load is

$$v_c = \frac{V}{bd} = 1.9\sqrt{f'_c} + 2500\,\frac{p\,Vd}{M} \leq 3.5\sqrt{f'_c} \qquad (3)$$

The allowable stress for working stress design is approximately one-half the stress given by Equation 3. The allowable stress for ultimate strength design is that of Equation 3 with some reduction by load factors and factors of uncertainty.

In design situations where the allowable stress is exceeded some type of web reinforcement is required. With the opening of a diagonal crack, the shear reinforcement acts in tension to carry load from one side of the crack to the other. The design criteria for web reinforcement is derived from the "truss analogy," which relates the behavior of the reinforced concrete member to that of a Warren truss. The stirrups are considered to act as tension diagonals while the concrete is assumed to act as compression diagonals and top chord, and the longitudinal steel to act as the bottom chord. It is also assumed that there is a diagonal tension crack extending to a depth jd above the tension steel.

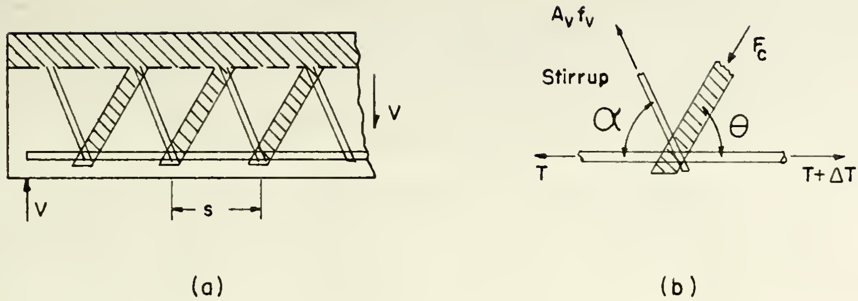


FIGURE 3. THE TRUSS ANALOGY

The strength of a beam with web reinforcement is derived using the truss analogy in the following manner:

θ = inclination of compression diagonal

α = inclination of stirrups

s = horizontal spacing of stirrups

V = total shear

V_c = shear assumed to be carried by concrete

V_s = shear assumed to be carried by stirrups

A_v = area of a stirrup

f_v = stress in a stirrup

For a condition of equilibrium,

(1) summation of vertical forces yields

$$F_c = \frac{A_v\ f_v \sin \alpha}{\sin \theta}$$

and

(2) summation of horizontal forces yields

$$\Delta T = A_v f_v \cos\alpha + F_c \cos\theta$$

or

$$\Delta T = A_v f_v \cos\alpha + A_v f_v \sin\alpha \frac{\cos\theta}{\sin\theta}$$

With the assumption of constant shear,

$$\Delta T = \frac{\Delta M}{jd} = \frac{Vs}{jd}$$

The result of equating the two expressions for ΔT is:

$$A_v = \frac{V s}{f_v\, jd\, (\cos\alpha + \frac{\sin\alpha}{\tan\theta})}$$

If the diagonal crack is assumed to occur at the inclination of $\theta = 45^\circ$, then the expression may be simplified:

$$A_v = \frac{V s}{f_v\, jd\, (\cos\alpha + \sin\alpha)} \qquad (4)$$

Since the uncracked concrete section carries a portion of the shear, V is replaced by $V_s = V - V_c$. With the additional definitions:

$$r = \frac{A_v}{bs \sin\alpha} \qquad \text{(stirrup ratio)}$$

$$K = (\sin\alpha + \cos\alpha)\sin\alpha \qquad \text{(stirrup efficiency)}$$

$$v_s = \frac{V_s}{bjd}$$

Equation 4 may be rewritten as:

$$rbs \sin \alpha = \frac{v_s \; bjd}{f_v \; jd \; (\cos \alpha + \sin \alpha)}$$

or

$$v_s = rKf_v \qquad (5)$$

For the case of vertical stirrups ($\alpha = 90^{\circ}$), $K = 1$, and Equation 5 becomes

$$v_s = r \; f_v$$

For purposes of design, the allowable shear stress for a cross-section with web reinforcement is the sum of the permissible shear stresses in the concrete and the steel, or $v = v_c + v_s$. This design procedure has been used for a long time and has worked well. However, it does not provide a rational explanation of beam behavior, and it must be subjected to certain restrictions to insure that the stirrups yield before failure.

Discussion of Design Philosophy

In the foregoing discussion of design principles it should be mentioned that the loads to be carried by a member are considered to be of a given magnitude, are applied in given locations, and are assumed to act as though they were applied slowly. However, the internal loads which are calculated for a statically applied load are not representative of the internal loads which would exist if the same load were

applied dynamically. Obviously, the momentum of the load itself will produce internal loads above the static values. The live loads on a bridge are dynamic in nature, and it is therefore necessary to account for the dynamic or impact effects. This is done usually by increasing the live load by some percentage in accordance with an impact factor, which has been developed through experience.

There are various impact formulas which are expressed as functions of the length of the member being designed, but their theoretical foundations are somewhat vague. The use of impact factors has produced safe designs as can be seen from experience although the impact factors may not have been very accurate in the assessment of dynamic loads.

When the consideration of fatigue is relevant, the maximum and minimum load carried by a structural member must be established in addition to the number of load repetitions to which that member will be subjected during its service life. For the case of a rural bridge which carries, perhaps, one truck (maximum design loading) per hour, 115 years will elapse before the maximum design load has been repeated one million times. On the other hand, an urban expressway bridge carrying, for example, 100 trucks per hour will be subjected to one million repetitions in only 1.15 years. It can be seen that fatigue stresses would be of lesser importance for the rural bridge. A more rational design of bridges would result if traffic load surveys were utilized.

Review of Literature

There have been several comprehensive reviews written in the last few years that consider the shear strength of reinforced concrete beams subjected to static loads. An excellent review of the historical development was written by Dr. Eivind Hognestad in 1951[10]. This review was updated by the Joint ACI-ASCE Committee 326[2] in 1962 and by the MSCE thesis of William Harvey[9] in 1964. It would seem that it is unnecessary to undertake such a review of this nature, as these reviews are readily available. However, it is interesting to note that previous studies of shear strength in reinforced concrete members have usually been accomplished using static loading.

The problem of fatigue of concrete, particularly in reinforced concrete members, has been investigated previously, but the scope of research has been relatively limited. A comprehensive historical review of research concerning fatigue of concrete was written by Nordby[16] in 1958. Nordby subdivided the research into six categories for the purpose of discussion: fatigue in compression, fatigue in flexure, fatigue in tension, fatigue in bond, fatigue in reinforced concrete, and fatigue in prestressed concrete. It was noted that most research prior to 1958 had been of an exploratory nature and provides a basis for further investigation. The ACI Committee 215 published an excellent bibliography in 1960[1] which provides a summary of research on fatigue of concrete from 1898 to 1958.

Reinforced concrete members can fail under repeated loading in many different ways just as under static loading. However, identical specimens may fail differently depending on whether they have been subjected to static or repeated load. As a result, certain modes of failure are more vulnerable to fatigue damage, and the relative factor of safety of a concrete structure under repeated loading will differ from static loading, which is the primary basis for current design practice.

For a beam that is weaker in shear than in flexure and is subjected to repeated loading, the failure mode may occur in any of the following ways: (1) diagonal cracking, (2) destruction of the compression zone, (3) splitting along the longitudinal reinforcement, (4) fatigue of the reinforcement, and (5) bond failure. The fatigue strength of reinforced concrete members is a complex subject. The mode of failure of the beam being considered will be influenced by the same factors believed to affect the shear strength of beams subjected to static loads (i.e., concrete strength, amount of longitudinal reinforcement, amount of web reinforcement, and shear span to depth ratio). In addition, the maximum value of the repeated load, the range of the repeated loading, and the rate of loading will all affect the behavior of the member. Generally, a beam weak in shear and subjected to a repeated load will crack diagonally with the crack progressing toward the compression zone until the compression zone is destroyed.

In a study by Chang and Kesler[6] it was found for beams of small cross section (4 inches x 6 inches) which were weak in shear and subjected to repeated loading that if a beam did not crack diagonally, it was not damaged by the repeated loading so far as the initial diagonal tension cracking load was concerned. In another study involving similar beams which were weak in flexure and subjected to repeated loading[7] it was found that the magnitude of the repeated load determined the mode of failure. Low magnitude loads generally produced fatigue failures in the reinforcement, while high magnitude loads resulted in shear failures.

Stelson and Cernica[17] have suggested that shearing stresses may be more critical than compressive stresses in beams subjected to repeated loads. In their study of eleven identical beams all failed in diagonal tension even though the unit shear stress at design load (according to the 1956 edition of the ACI Code) was only 82 percent of the allowable shear stress. It was found in a study by Verna and Stelson[18] that the fatigue resistance of concrete members was greater for concrete of higher strengths. The results of these investigations suggest that the behavior of beams which are weak with respect to diagonal tension need further investigation. In all of the research concerning beams that were weak with respect to diagonal tension and were subjected to repeated loads the specimens were tested as simply supported members.

Investigations which have been recently reported by Magura and Hognestad[13] have indicated that the use of repeated loading can successfully be employed in the laboratory to duplicate the loading situation as found in the field. This project was able to correlate the behavior of prestressed concrete bridge girders which were tested in the laboratory using repeated loads with the behavior of identical girders which were tested in the AASHO Road Test.

There has apparently been very little research using repeated loads of beams that are weak with respect to diagonal tension. To the author's knowledge, there have not been any similar tests that have used continuous members. In summary, the four principal modes of failure which have been included in previous investigations are: (1) Diagonal tension, (2) Compression of the concrete, (3) Bonding of the concrete to steel, and (4) Brittle failure of tensile steel. Of these modes, the first and third seem to be most affected when repeated loads are applied.

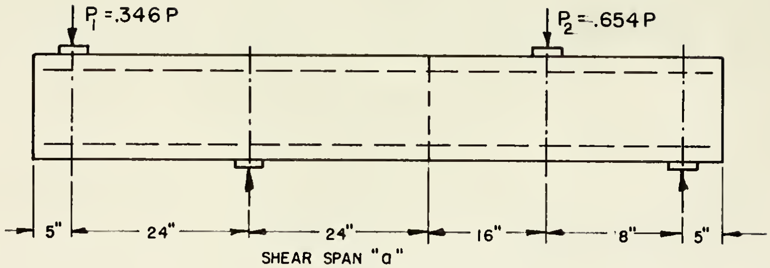
PURPOSE OF STUDY

The objective of this study was to observe the behavior of beams of different shear span-to-depth ratio with varying amounts of web reinforcement which are subjected to repeated loads. The magnitude of the repeated loads was varied to determine the effect on the failure mode of the specimens. These observations were to be compared with tests on similar rectangular specimens which had been loaded statically.

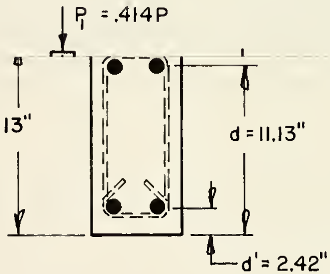
TEST SPECIMENS AND PROCEDURES

All specimens were simply-supported with one overhang to simulate the conditions of an interior support of a continuous member. A load (P_1) was applied to the cantilever portion, and a second load (P_2) was applied to the beam between the supports. These two loads were imposed on the beam through a steel I-section which rested on steel rollers with seats. The desired ratio of P_1 to P_2 and, consequently, the ratio of maximum negative moment to maximum positive moment, was achieved by positioning the applied load on the I-section. The loads, the shear and moment diagrams, and the details and dimensions of the specimens may be found in Figure 4 and Table 1.

All beams had a 6 inch by 13 inch rectangular cross section. The major variables were the length of the shear span "a," the amount of web reinforcement within the shear span, and the magnitude of the repeated load. In order to restrict failure to the shear span "a," an excessive amount of web reinforcement was placed in the cantilever portion and outside of the load P_2. The longitudinal reinforcement in all specimens consisted of two No. 6 bars in the top and two No. 5 bars in the bottom. The maximum negative moment, M_1, was maintained at 1 1/2 and 2 times the maximum positive moment, M_2.



SERIES I



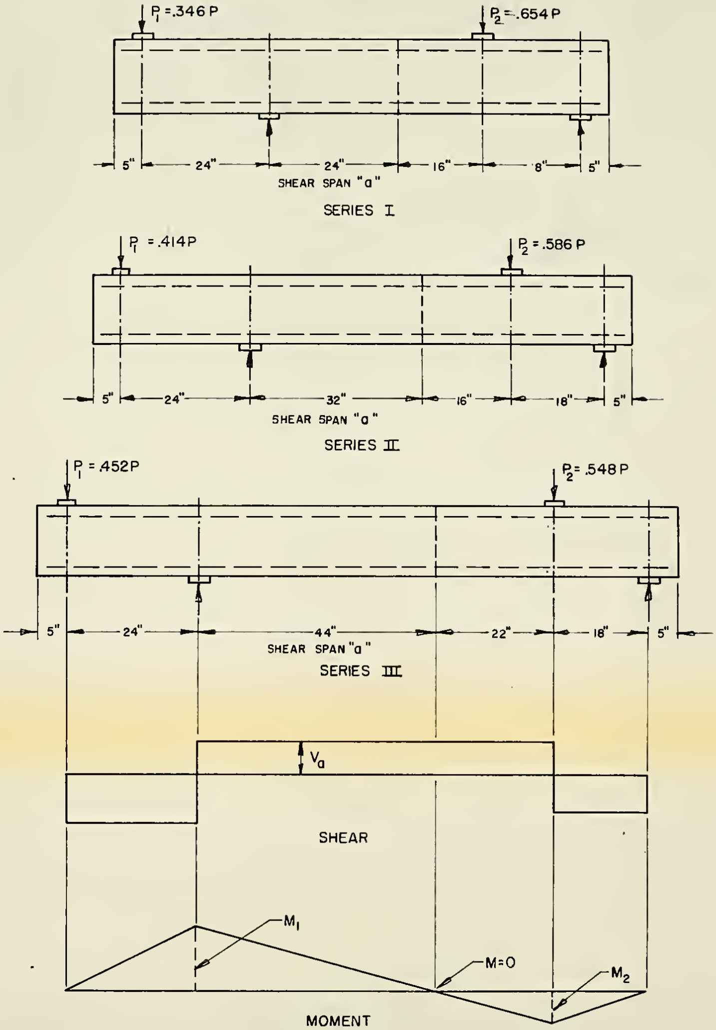
CROSS SECTION

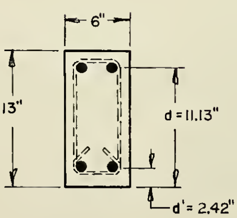
P_2 = .586 P

SERIES	I	II	III
M_1 / M_2	1.5	2	2
a / d	2.16	2,88	3.96
V_a	,346 P	,310 P	.247 P
M_1	8,30 P	9.94 P	10.85 P

All Beams: 2 – No.6 Top
2 – No.5 Bottom

FIGURE 4 DETAILS OF SPECIMENS





CROSS SECTION

SERIES	I	II	III
M_1 / M_2	1.5	2	2
a/d	2.16	2.88	3.96
V_a	.346 P	.310 P	.247 P
M_1	8.30 P	9.94 P	10.85 P

All Beoms: 2 – No.6 Top
2 – No.5 Bottom

FIGURE 4 DETAILS OF SPECIMENS

e ive th	Age (Days)	Split Tension (psi)	$E \times 10^6$ (psi)
	56	-	4.87
	14	566	4.70
	14	422	4.40
	14	452	4.58
	10	468	3.32
	14	483	4.27
	28	453	4.47
	14	482	3.95
	14	569	4.15
	14	480	4.42
	21	465	4.38
	21	512	4.35
	28	580	5.22
	21	560	4.67
	21	580	4.77
	28	545	4.82

#5 bars.

Table 1. Properties of Test Specimens

Beam Designation	d	d'	a	a/d	Web Reinforcement Size and Spacing	r (A_v/bs)	rf_{vy} (psi)	Concrete Compressive Strength (psi)	Age (Days)	Split Tension (psi)	E x 10^6 (psi)
I BF-1	11.13"	2.42"	24"	2.16	-	-	-	6750	56	-	4.87
I BF-2	"	"	"	"	-	-	-	5488	14	566	4.70
I BF-3	"	"	"	"	-	-	-	5088	14	422	4.40
I BF-4	"	"	"	"	-	-	-	5825	14	452	4.58
I BF-5	"	"	"	"	-	-	-	4000	10	468	3.32
I BF-6	"	"	"	"	-	-	-	4870	14	483	4.27
I BF-7	"	"	"	"	-	-	-	5425	28	453	4.47
II BF-1	"	"	32"	2.88	-	-	-	4400	14	482	3.95
II BF-2	"	"	"	"	-	-	-	5580	14	569	4.15
II BF-3	"	"	"	"	-	-	-	5260	14	480	4.42
III BF-1	"	"	44"	3.96	-	-	-	5050	21	465	4.38
III BF-2	"	"	"	"	-	-	-	4860	21	512	4.35
III BF-3	"	"	"	"	-	-	-	5630	28	580	5.22
II BFR-1	"	"	32"	2.88	#4 Wire @ 3-1/2"	.00381	87.6	5460	21	560	4.67
II BFR-2	"	"	"	"	"	"	"	5390	21	580	4.77
II BFR-3	"	"	"	"	"	"	"	5780	28	545	4.82

All specimens have as longitudinal reinforcement: Top - 2 #6 Bars and Bottom - 2 #5 bars.

The percentage of longitudinal reinforcement, p, is 0.01318 for all specimens.

The modulus of elasticity for the concrete is the Initial Tangent Modulus.

Materials

Concrete Mix

All concrete was made with Type I Portland cement. The concrete strength was intended to be maintained at 5000 psi at the age of 14 days, but varied from approximately 4000 to 5800 psi. The minimum age of the specimens at the beginning of each test was 10 days, with most specimens being tested at either 14, 21, or 28 days. The proportions of the mix by saturated-surface-dry weight were 1:2.9:3.73 (cement to fine aggregate to coarse aggregate) with a water-cement ratio (w/c) of .548 by weight and a cement factor of 6.48 sacks/yd^3.

Aggregates

The aggregates used were purchased from Western Indiana Aggregates Corporation, Lafayette. The coarse aggregate was a natural gravel (Western Indiana's No. 8) of 1 inch maximum size. In the laboratory it was separated into two sizes to minimize segregation during handling. By Fuller's Maximum Density Curve, 48 percent of No. 4 to 1/2 inch was combined with 52 percent of 1/2 to 1 inch, by weight. Average properties of the fine and coarse aggregate were as shown below.

	Sp. Gr.*	Absorption	Fineness Modulus
Fine	2.83	1.26%	2.87
Coarse	2.65	1.37%	----1" Max. Size

*Based on saturated-surface-dry weight.

Gradation of Fine Aggregate

Sieve Size	Percent Retained
No. 4	1.4
8	16.6
16	36.8
30	48.0
50	87.8
100	96.6

Reinforcing Steel

The longitudinal reinforcing steel was a high strength steel with the average properties shown in Table 2. The No. 5 and No. 6 deformed bars were rolled from the same heat. The properties shown are for coupons selected at random. A representative stress-strain curve is shown in Figure 43, Appendix A. The deformations met the requirements of ASTM-A305.

Table 2. Properties of Longitudinal Reinforcement Steel

Yield Stress	72.1 ksi
Ultimate Strength	113.3 ksi
Modulus of Elasticity	30.0×10^6 psi
Elongation in 8"	16.6%

The No. 2 plain bars used for stirrups in the overhang and the 18 inch interior span were of hard grade steel with an average yield strength of 52,000 psi.

Stirrups in the critical shear span "a" were made of a very soft No. 4 wire (diameter = .224 inch) which was obtained from the Continental Steel Corporation, Kokomo, Indiana. Several coupons were randomly selected to determine the properties shown in Table 3. A representative stress-strain curve is shown in Figure 44, Appendix A.

Table 3. Properties of Soft Web Reinforcement

Yield Stress	23.0 ksi
Ultimate Strength	43.7 ksi
Modulus of Elasticity	27.2×10^6 psi

Fabrication and Curing

All specimens were cast in 3/4 inch plywood forms which were made of treated plywood to prevent deterioration from the repeated wetting and drying. The forms are shown partially assembled in Figure 5. The side braces were made of 1 inch by 1 inch angle iron and were bolted to the base of the forms. Tie rods were used at both ends to hold the end bulkheads in place.

The reinforcement was wired into a rigid cage with the stirrups being wrapped around the longitudinal steel. A minimum of 1.4 inches clear between the longitudinal bars was maintained by wiring the bars to the stirrups. The concrete cover was a minimum of 1 1/2 inches to the longitudinal

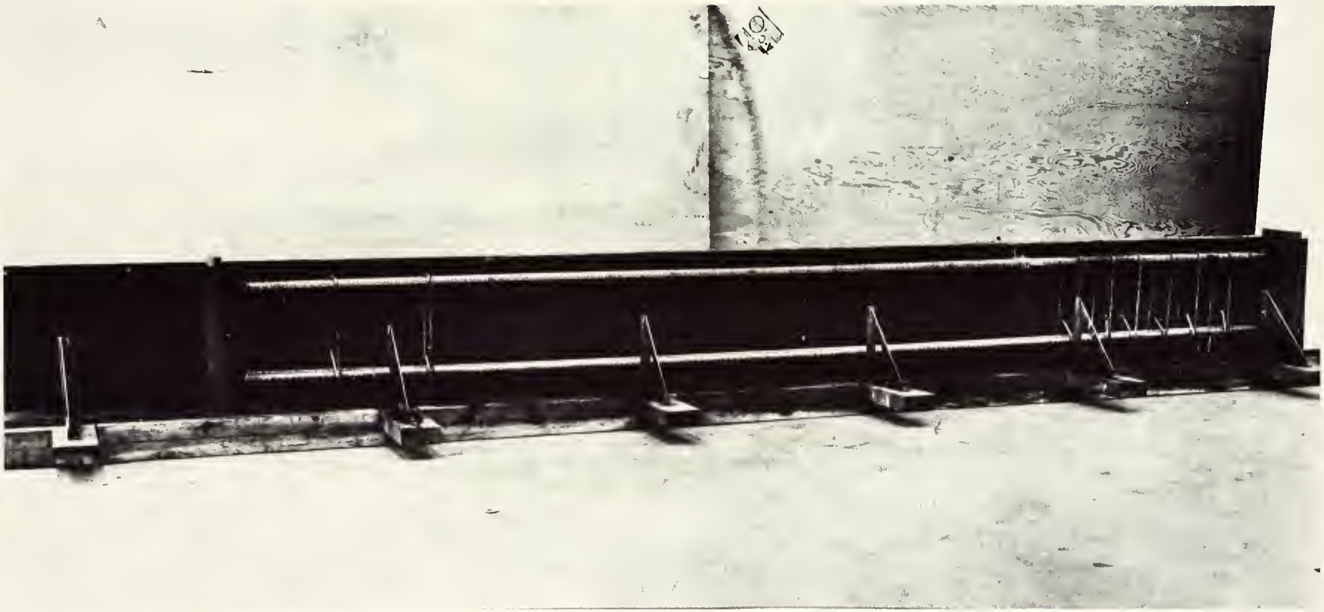


FIGURE 5. FORMS PRIOR TO CASTING

steel in all locations. The cage was supported by rigid steel chairs to provide 2 inches clear cover on the bottom of the specimens. The lateral position of the cage was maintained by using wood blocks which were removed as the concrete was placed.

The concrete was mixed in a stationary rotating drum mixer with a maximum capacity of eleven cubic feet. The quantities of materials were weighed before mixing was started. Six control cylinders (6 inches by 12 inches) were cast with each specimen. Internal vibration was used in the placing of the concrete.

Several hours after placement of the concrete the top of the forms and the cylinder molds were covered with moist burlap. A sheet of plastic was then placed over the burlap. The forms were removed after three days, and the specimens and control cylinders were stored in a moist curing room until two days before they were tested.

Instrumentation and Testing Procedures

An Amsler Hydraulic Pulsator was used to actuate a remote hydraulic jack. The jack was bolted to a reinforced concrete frame which was post-tensioned to the laboratory floor. The pulsator produced a sinusoidally varying oil pressure which produced a load at the jack at the rate of 250 cycles per minute. A view of the test frame and details of the setup may be found in Figures 6 and 7 respectively. The pulsator was equipped with a shutoff device which was

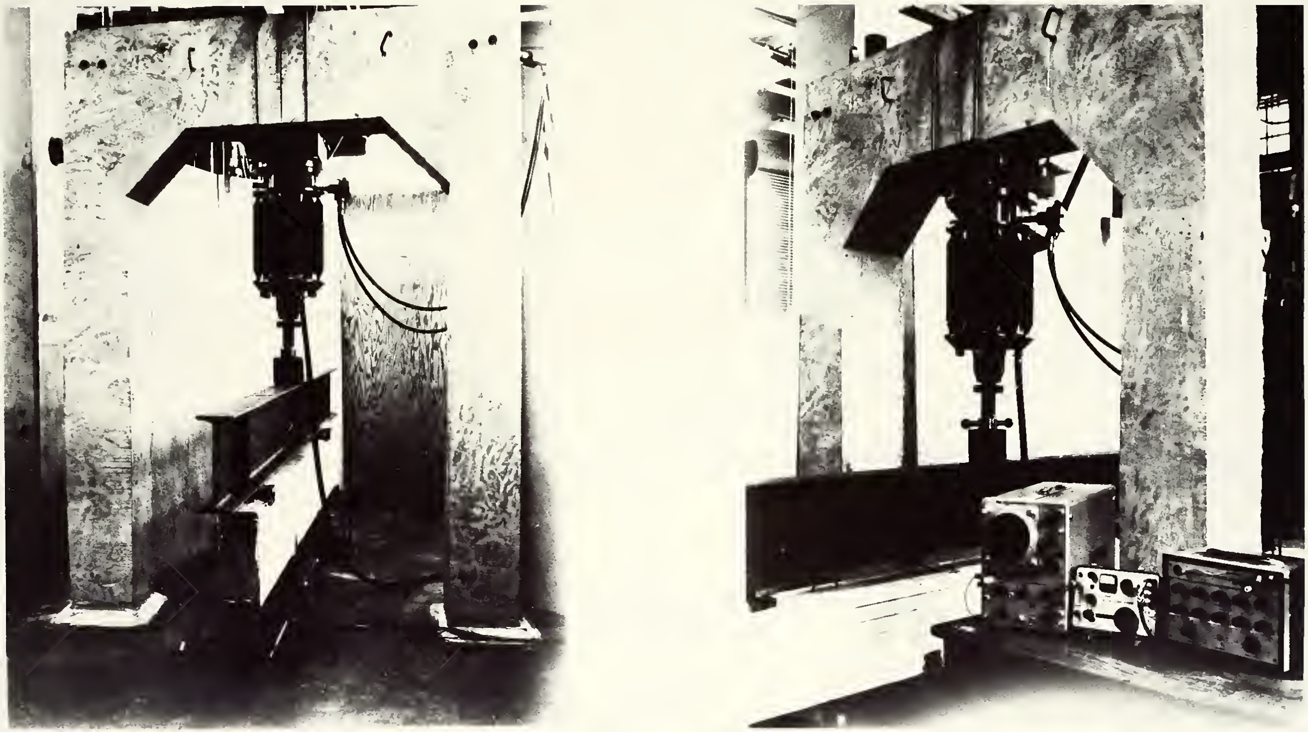
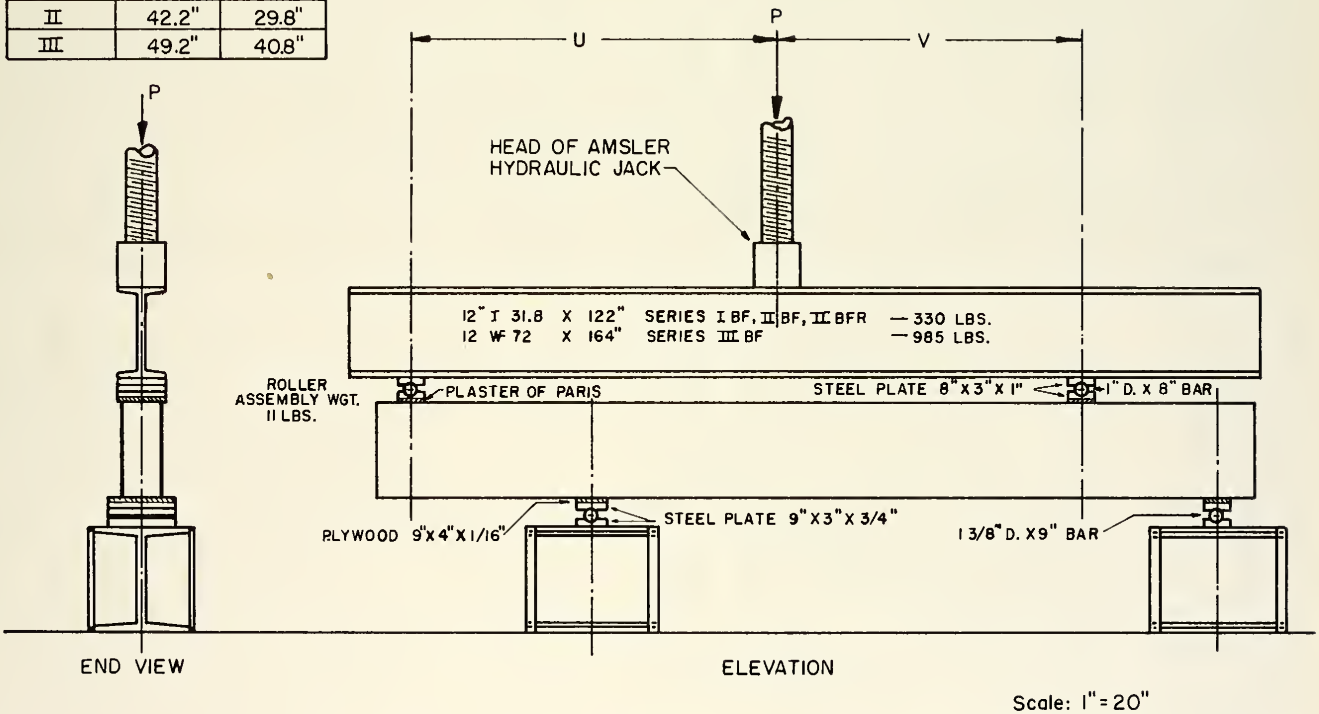


FIGURE 6. TEST SETUP

SERIES	U	V
I	41.9"	22.1"
II	42.2"	29.8"
III	49.2"	40.8"



Scale: 1" = 20"

FIGURE 7 DETAILS OF TEST SETUP

actuated by a drop in load level or by a sudden shock. Thus, tests could be run without continuous surveillance. Details of the reinforced concrete frame may be found in Figure 45, Appendix B.

The steel strains were measured with the use of foil type electric strain gages (Budd Metalfilm C6-141B). Prior to preparation of the reinforcement cage, the gages were affixed to the reinforcement at prepared locations using Eastman 910 adhesive. The gages and their lead wires were waterproofed with an epoxy compound (Budd GW-7). Strain in the longitudinal reinforcement was measured at the point of maximum moment. Strains were also measured on some of the stirrups located within the critical shear span. The actual location of each of the gages is indicated on the crack pattern sheets.

Compressive strains in the concrete were measured at 3 1/2 inches from the support. Paperback SR-4 wire gages (Baldwin-Lima-Hamilton Type A-1-S6) were cemented to the concrete surface using Duco cement. The concrete surface had been smoothed with an emery stone, cleaned with acetone, and sealed with Duco cement one day before application of the gages.

The strain signals from the specimen during a test were comprised of a static component and a dynamic component. In order to measure the strain levels without interruption of a test, a Budd Model P-350 digital read-out strain indicator was used in conjunction with a Tektronix Model 515A

oscilloscope. The static component of the strain signal was assessed with the indicator while the dynamic component was read from the calibrated oscilloscope. A switching and balancing unit was employed so that several gages could be read quickly.

The sides of each beam were painted white with a mixture of Plaster of Paris and water. Grids were then drawn to facilitate tracing and drawing of the crack pattern. The crack patterns were later recorded and the specimens were photographed.

Three control cylinders were tested in compression for each specimen with the remainder being tested in split-tension. Two of the compression cylinders were used to determine the modulus of elasticity of the concrete with the aid of a ten inch extensometer attached to the cylinders.

Once the concrete cylinder strength was known, an estimate of the ultimate strength of the specimen with respect to shear was made using the expressions of Moody[14], Morrow[15], and the ASCE-ACI Joint Committee 326[2]. A percentage of the predicted ultimate load was then taken as the maximum load to be repeated. For beams without web reinforcement an average of the estimate suggested by Moody and Morrow was used to predict the ultimate load. For beams with web reinforcement, the ultimate load was taken as 12 per cent greater than the ultimate load suggested by ASCE-ACI Committee 326 since similar specimens tested by Harvey[9] indicated a slightly greater strength. The minimum load to

be repeated was maintained at three kips to prevent impact on the specimen.

For the first cycle of load, the specimen was loaded statically in increments of one to five kips to the maximum load to be repeated. After each increase in load, the crack penetration was traced and the load marked on the beam. The strains were also read at this time. After the maximum load had been reached and the cracks noted, the beam was unloaded, and the repeated load was applied. While the specimen was being subjected to the repeated load, strain readings were taken at various stages, and the growth of cracks was noted. Most of the specimens tested failed during the application of the repeated load. However, there were a few specimens which exhibited very little damage caused by at least one million cycles of load. These specimens were terminated and loaded statically to failure.

TEST RESULTS

The pertinent test results have been summarized in Table 4. Photographs of the beams after testing are shown in Figures 8 through 11. Strain measurements for the first load cycle of most beams are presented graphically in this section. The complete strain data for all specimens may be found in tabular form in Appendix D. Scale drawings of the beams which show the crack patterns and the locations of the strain gages are included. A brief description of each test is given as a record of the observed behavior. The loads reported are total applied loads, which do not include the dead weight of the specimen and the weight of the loading apparatus. Figure 15 is a pictorial explanation of the way in which the specimens are presented on the crack pattern sheets.

The occurrence of the critical diagonal tension crack in beams without web reinforcement was easily determined in most situations. However, in beams with web reinforcement and in some of the longer specimens the formation of the diagonal tension crack was more difficult to detect. For this reason the diagonal cracking load is herein defined as the load at which the critical diagonal crack was observed to cross the neutral axis, using the cracked-section theory.

Ultimate Shear Stress v_u*** (psi)	Mode of Failure##	Remarks
297	T / S.C.	
168	F / D.T.	
223	F / S.C.	
308	T / D.T.	
215	T / D.T.	
358	T / D.T.	
166	F / F.R.	
154	F / D.T.	
144	F / D.T.	
204	T / D.T.	
118	F / F.R.	
98	F / F.R.	
178	T / D.T.	
193	F / F.R.	
165	F / F.R.	
223	F / F.R.	

apparatus.)

nce of fatigue failure.
repeated load;
orcement.

Table 4. Summary of Test Results

Beam Designation	Applied Load P_{max} (kips)	Applied Load P_{min} (kips)	Percent of P_f*	Fatigue Life N (cycles)	Diagonal Cracking Load P_c** (kips)	Ultimate Load P_u** (kips)	Shearing Stress At Diagonal Cracking v_c*** (psi)	Ultimate Shear Stress v_u*** (psi)	Mode of Failure##	Remarks
I BF-1	20.5	3.0	28.5	9,966,600t#	55.0	57.1	285	297	T / S.C.	
I BF-2	32.5	3.0	50	132,000	30.0	32.5	156	168	F / D.T.	
I BF-3	43.0	3.0	70	15,100	40.0	43.0	207	223	F / S.C.	
I BF-4	27.0	3.0	40	3,500,000t	56.2	59.4	292	308	T / D.T.	
I BF-5	31.5	3.0	60	868,300t	31.3	41.5	161	215	T / D.T.	
I BF-6	35.5	3.0	60	2,038,500t	34.0	69.3	176	358	T / D.T.	
I BF-7	32.0	3.0	50	2,938,000	32.0	32.0	166	166	F / F.R.	
II BF-1	33.0	3.0	70	1,500	30.0	33.0	139	154	F / D.T.	
II BF-2	31.0	3.0	60	8,800	31.0	31.0	144	144	F / D.T.	
II BF-3	25.0	3.0	50	3,000,000t	25.0	44.0	116	204	T / D.T.	
III BF-1	32.0	3.0	70	1,290,600	31.5	32.0	116	118	F / F.R.	
III BF-2	26.5	3.0	60	3,864,400	26.5	26.5	98	98	F / F.R.	
III BF-3	37.0	3.0	80	88,000t	37.0	48.0	137	178	T / D.T.	
II BFR-1	41.5	3.0	70	1,212,600	35.0	41.5	162	193	F / F.R.	
II BFR-2	35.5	3.0	60	1,846,400	35.0	35.5	162	165	F / F.R.	
II BFR-3	48.0	3.0	80	296,400	37.0	48.0	149	223	F / F.R.	

* Predicted ultimate load
** Total applied load. (Does not include dead weight of specimen and loading apparatus.)
*** Average shearing stress v = V/bd, in critical shear span.

\# t signifies rests in which repeated load was terminated before the occurrence of fatigue failure.
\## F - Fatigue failure; T - Tested statically to failure after termination of repeated load; D.T. - Diagonal Tension; S.C. - Shear Compression; F.R. - Fatigue of Reinforcement.

BEAM I BF-1

BEAM I BF-2

BEAM I BF-3

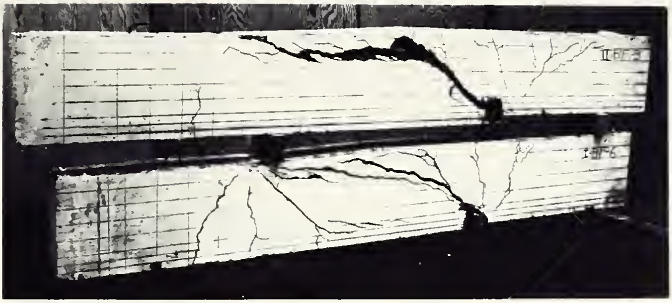
FIGURE 8. BEAMS AFTER TEST - SERIES I BF

BEAM I BF - 4

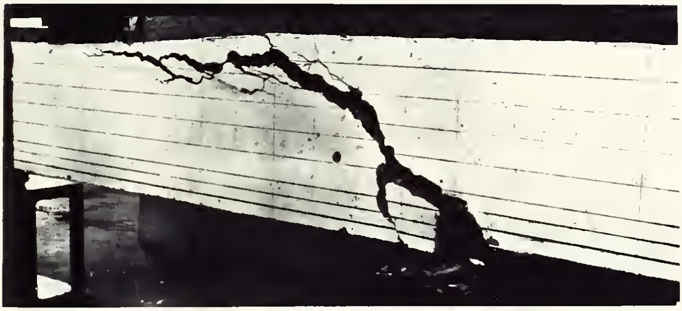
BEAM I BF - 5

BEAM I BF - 7

FIGURE 9. BEAMS AFTER TEST - SERIES I BF



BEAMS II BF-3 & I BF-6

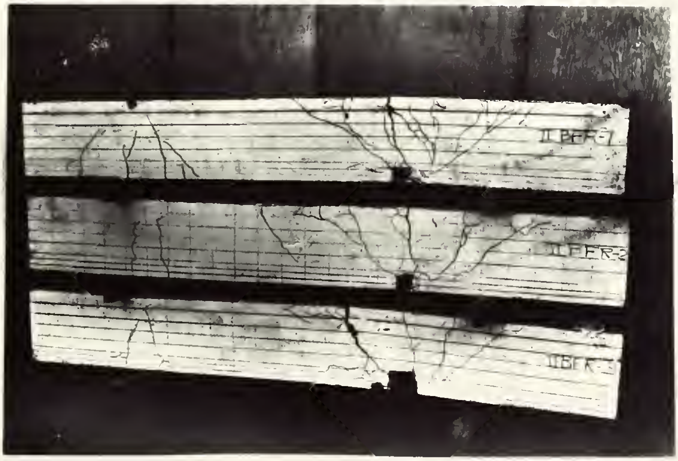


BEAM II BF-1

BEAM II BF-2

FIGURE 10. BEAMS AFTER TEST - SERIES I BF & II BF

BEAMS III BF-1, III BF-2, III BF-3



BEAMS II BFR-1, II BFR-2, II BFR-3

FIGURE 11. BEAMS AFTER TEST - SERIES III BF & II BFR

The depth of the cracked section neutral axis was nominally four inches from the compression face.

Series I BF

The major variable for the first series was the maximum amplitude of the repeated load. The maximum repeated load was chosen from the predicted ultimate load for each specimen and ranged from nominally 30 to 70 percent of ultimate. The specimens were 7'-8" in length and had a shear span-to-depth ratio (a/d) of 2.16. The ratio of restraining moment to positive moment (M_1/M_2) was 1.5 (see Figure 4). There was no web reinforcement in the critical shear span. The corresponding beam in Harvey's report[9] is Beam IB. Load strain relationships for the first cycle may be found in Figures 12 and 13.

Beam I BF-1 (28.5% Ultimate)

This specimen was not loaded statically on the first cycle. After 100,000 cycles of loading three flexural cracks had penetrated to a depth of 8 inches from the compressive face. The maximum steel stress in the longitudinal reinforcement was 10,000 psi. After one million cycles of loading the flexural cracks had grown to a depth of four inches from the extreme fiber at the section of maximum moment. There was also the appearance of flexural cracks outside of the critical region. The maximum steel stress had increased to 20,000 psi. For the remaining applications of load there was no additional cracking. The maximum steel

stress increased progressively and finally indicated yielding. However, this may have been a failure of the strain gage. After almost ten million cycles of load the repeated load test was terminated. The specimen was then tested to failure statically. During the static test a diagonal crack appeared in the critical shear span and penetrated to within 1 inch of the compression face. The load was increased to 57.1 kips, and the specimen failed in shear-compression with a large section of the compression zone spalling off.

Beam I BF-2 (50% Ultimate)

Flexural cracks had penetrated to within 8 inches of the support at 9,400 cycles. These cracks increased 2 inches in length at 15,000 cycles and were accompanied by a diagonal tension crack which extended from 4 inches to 12 inches into the shear span. The diagonal crack grew in length with increasing repetitions until it extended from the support to the load point. The diagonal tension crack increased in width, and failure occured at 132,000 cycles. The maximum steel strain prior to failure was 680 $\pm$ 90 MII (19,000 $\pm$ 1500 psi). The expression MII denotes micro-inches per inch of strain. The static component of strain for this case is 680 MII with a dynamic variation during a cycle of loading of $\pm$ 90 MII. Thus, the maximum strain indicated is 720 MII while the minimum is 580 MII.

Beam I BF-3 (70% Ultimate)

This beam (and subsequent specimens) were loaded statically to the maximum amplitude of the repeated load for the first cycle. The first flexural crack appeared at P = 10 kips and penetrated to the level of the tension steel. After the load had been increased to 25^k, the flexure crack had penetrated to within 2 1/2 inches of the compression face. The steel and concrete strains were still linear as may be seen in Figures 12 and 13. Another flexural crack opened at 9 inches into the shear span and inclined toward the support at 25^k, and the beginning of a diagonal crack in the overhang was noted. With increases in load the critical diagonal crack progressed toward the support and into the shear span. At a load of 43^k, the diagonal crack was 19 inches into the shear span. The maximum steel strain was 1150 MII (32,500 psi), and the maximum concrete strain was 840 MII (3600 psi). The application of repeated loading brought additional cracking immediately. At 6500 cycles the diagonal crack had lengthened to the vicinity of the load point. There was also the formation of a second diagonal crack from just outside of the shear span to the level of the compression steel. A crack from near the support and along the compression steel appeared at 6500 cycles. By 9500 cycles these cracks had joined. The beam failed at 15,100 cycles with the spalling of two large pieces of concrete which were bounded by the diagonal crack.

Beam I BF-4 (40% Ultimate)

After the beam had been loaded to 26.5^k on the first cycle, there were three flexural cracks in the maximum moment region, which extended to within 8 inches of the compression face. After 1100 repetitions one of the original flexure cracks began to incline toward the support. At 48,000 cycles the flexural cracks had lengthened and penetrated to within 3 inches of the support. There were also two flexural cracks that progressed up toward the interior load point. However, after 77,500 cycles no additional cracking was noticed. The steel strain remained relatively constant at 850 $\pm$ 250 MII (25,000 $\pm$ 6500 psi) for the duration of the test, and the concrete strains were consistent at 350 $\pm$ 120 MII (1600 $\pm$ 550 psi). The repeated load was removed after 3.5 million repetitions. In the static retest there were very few additional cracks until the appearance of a critical diagonal crack at 56.2^k. The diagonal crack extended from the original flexural crack in the shear span to the interior load point. The load was increased to 59.4^k, and the beam failed suddenly with the formation of a second diagonal tension crack which occurred at the level of the compression steel and headed toward the load point.

Beam I BF-5 (60% Ultimate)

The first flexural cracks were similar to those of Beams I BF-2, 3, and 4. At a load of 31.3^k on the first cycle there were three flexural cracks in the maximum moment

region which had penetrated to within 4 inches of the compressive face. The cracks lengthened and were within 1 inch of the support after 50,000 cycles. The flexural crack in the shear span began to increase along the tension steel and extended across the shear span by 700,000 cycles to form the diagonal crack. During this process there were very little changes in the steel and concrete strain levels which were 520 ± 260 MII and 216 ± 150 MII, respectively. After 815,500 cycles the maximum amplitude was increased to 41.5^k, and a second diagonal crack was formed along the compression steel in the shear span. At 817,600 cycles the maximum amplitude was reduced to 34^k. The concrete strain had increased to 382 ± 220 MII at 839,000 cycles while the steel strain increased slightly to 554 ± 240 MII. Failure occurred at 868,300 cycles.

Beam I BF-6 (60% Ultimate)

Flexural cracks in the vicinity of maximum moment and under the interior load point had penetrated to within 4 inches of the compression face under a static load of 35.5^k. At a load of 34^k a diagonal crack had started to form in the critical shear span. After 14,000 cycles the diagonal crack extended across the shear span on the south side of the beam. The steel strain was 746 ± 500 MII before the strain gages failed at 14,000 cycles. The concrete strain in the extreme fibers reached a maximum of 325 ± 200 MII at the beginning of the repetitions and then decreased while the strain at 1 inch from the extreme fiber showed an increase during the

test. At 802,000 cycles there was some splitting along the tension steel, and the diagonal tension crack had penetrated to within 1/2 inch of the support. There were no noticeable changes in the cracks for the duration of the test which was stopped after 2,038,500 cycles. The beam was then loaded statically to failure. At a load of 55^k a new diagonal tension crack formed in the shear span slightly below the original crack. An additional diagonal crack opened beyond the shear span when the load was increased to $P = 58^k$. The beam continued to take additional load with an increase in the width of the diagonal crack until failure at $P = 69.3^k$.

Beam I BF-7 (50% Ultimate)

There were two flexural cracks that penetrated to within 3 inches of the support after the beam had been loaded to 30^k. At a load of 32^k a 3 inch flexural crack opened at a distance of 14 inches into the shear span. After 2000 cycles this crack inclined toward the support and had progressed to an inch from the compressive face. The diagonal cracking was accompanied by an increase in the concrete strain at the extreme fiber from 360 ± 240 MMI to 425 ± 240 MII. After 57,800 repetitions the diagonal crack had crossed the shear span. There were also two flexural cracks under the interior load point. The test proceeded with some additional cracking over the support. Failure occurred after 2,938,000 cycles with brittle fracture of both longitudinal bars at the location where they were crossed by the diagonal crack.

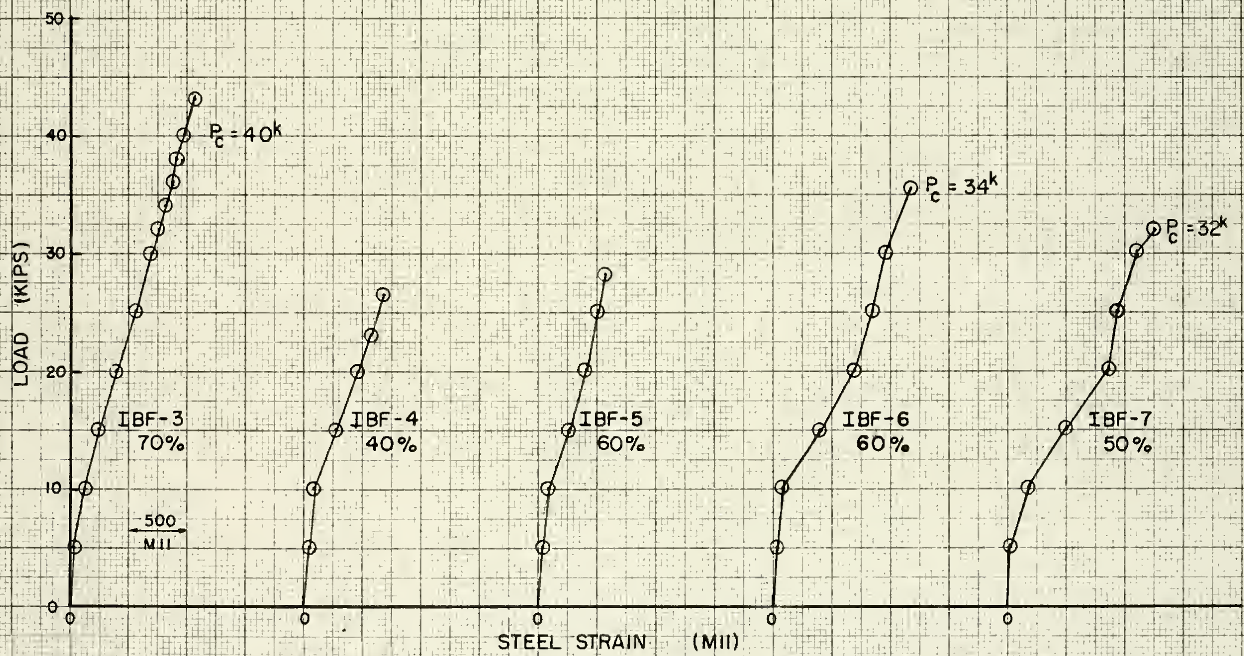


FIGURE 12. LOAD vs. STEEL STRAIN - SERIES IBF - FIRST CYCLE

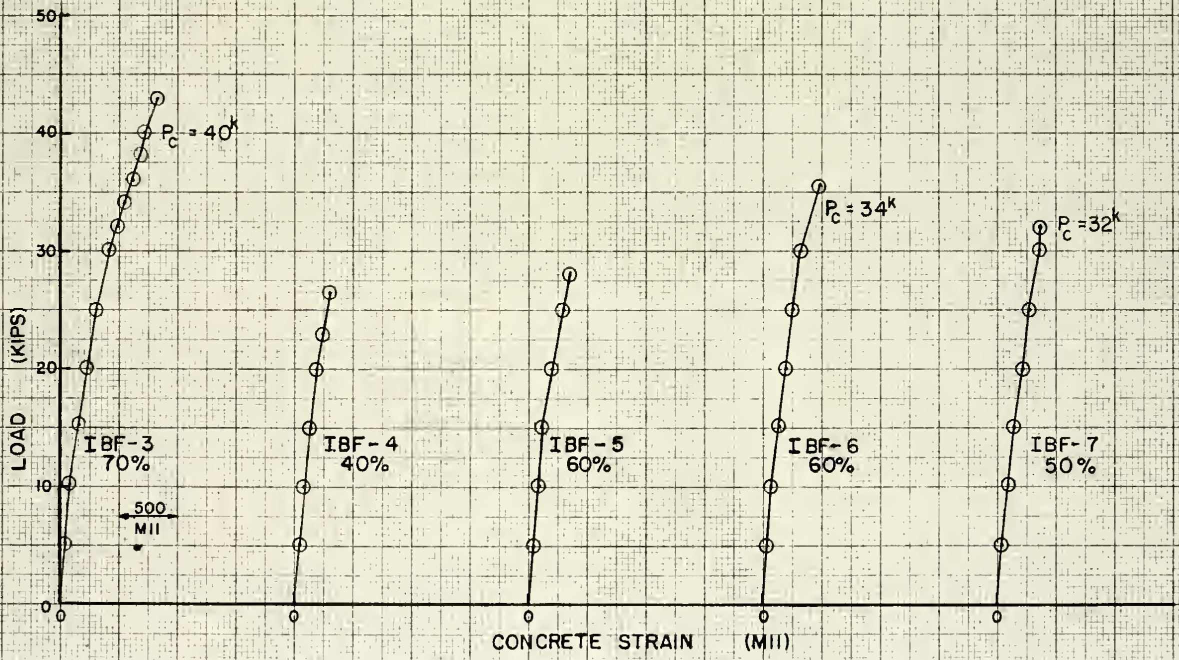


FIGURE 13. LOAD vs. CONCRETE STRAIN — SERIES I BF — FIRST CYCLE

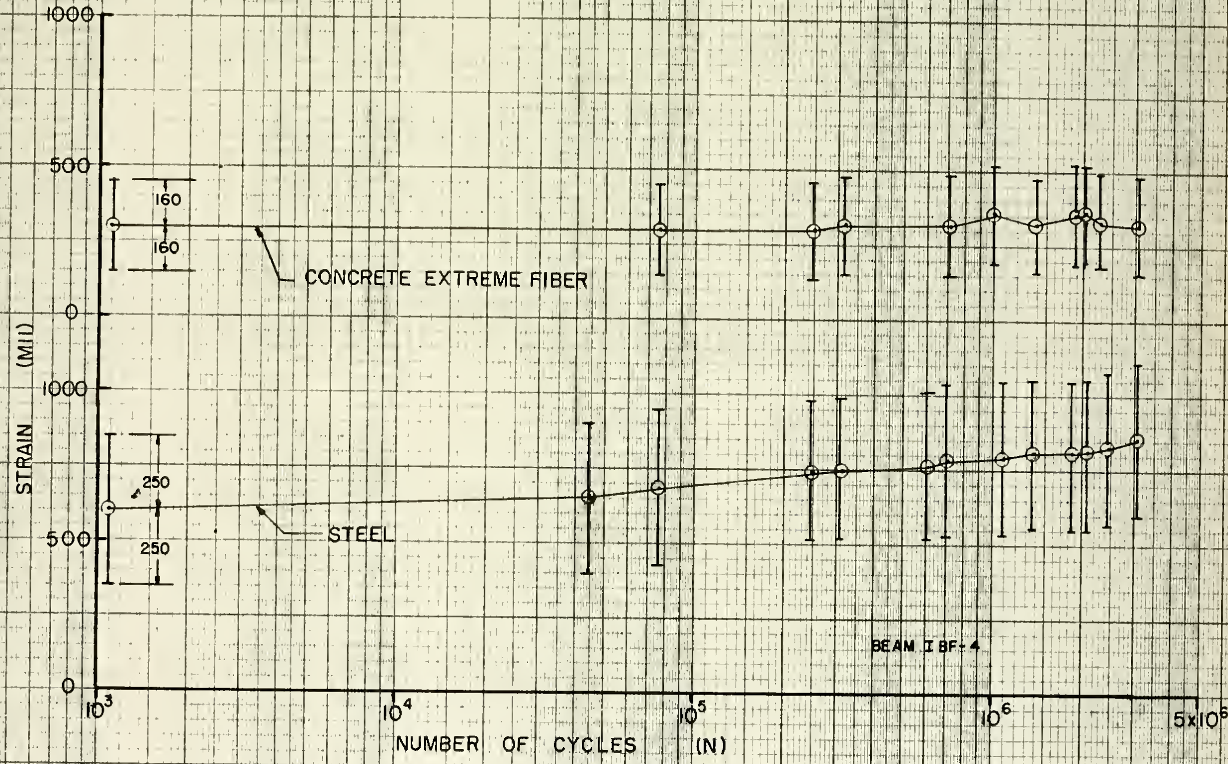


FIGURE 14. TYPICAL STRAIN vs. N RELATIONSHIP – SERIES I BF

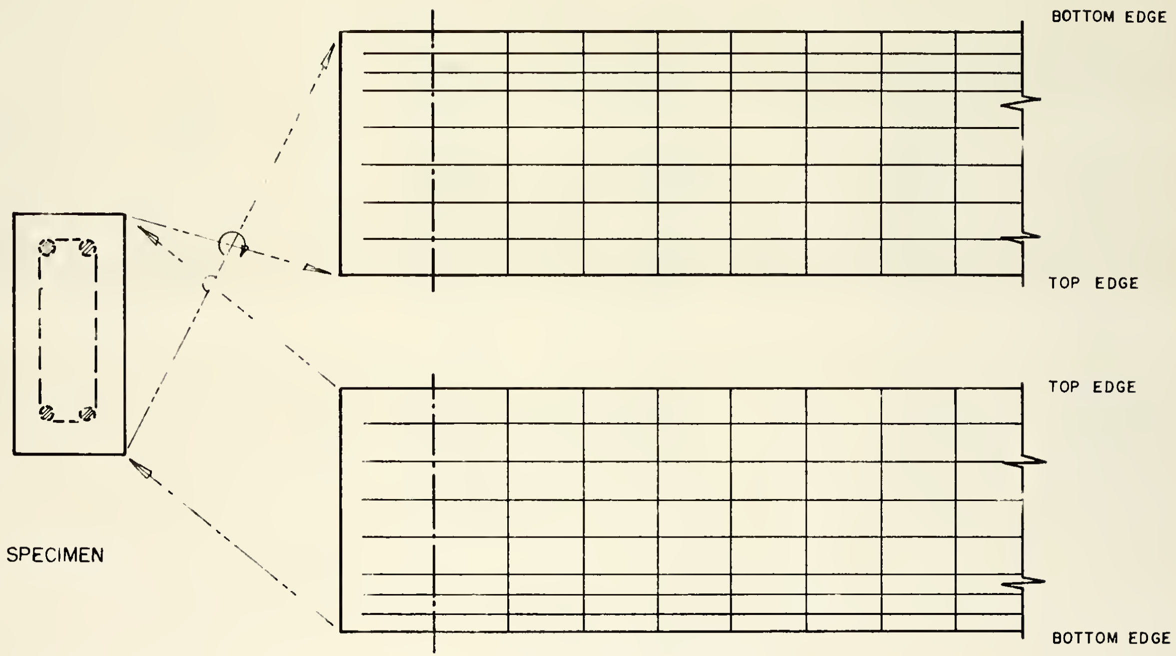
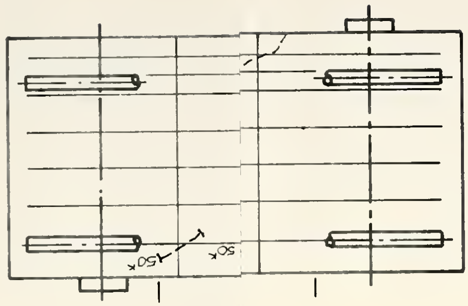
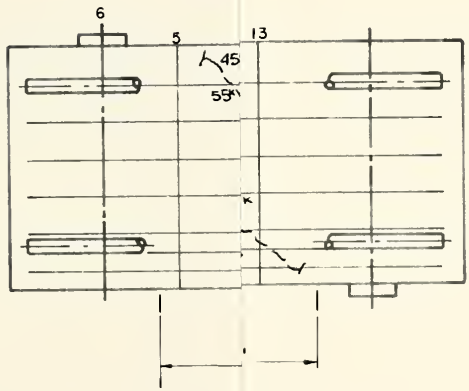


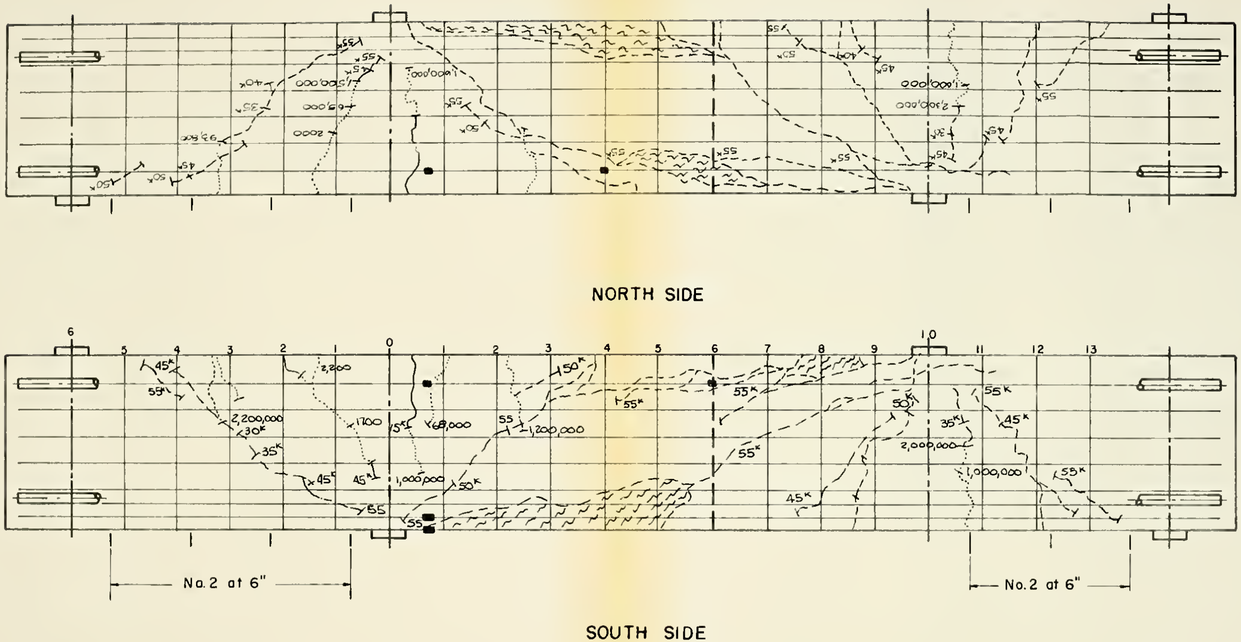
FIGURE 15. PICTORIAL REPRESENTATION OF SPECIMEN ON CRACK PATTERN SHEET





Foil Gage Locations
No. 6 Bar – load
" " –
" " –failure
" " –

SR-4 Gage Locat
0", 1" from



Foil Gage Locations:

No. 6 Bar — 3 1/2" from support (N)
" " — " " (S)
" " — 16" " " (N)
" " — 24" " " (S)

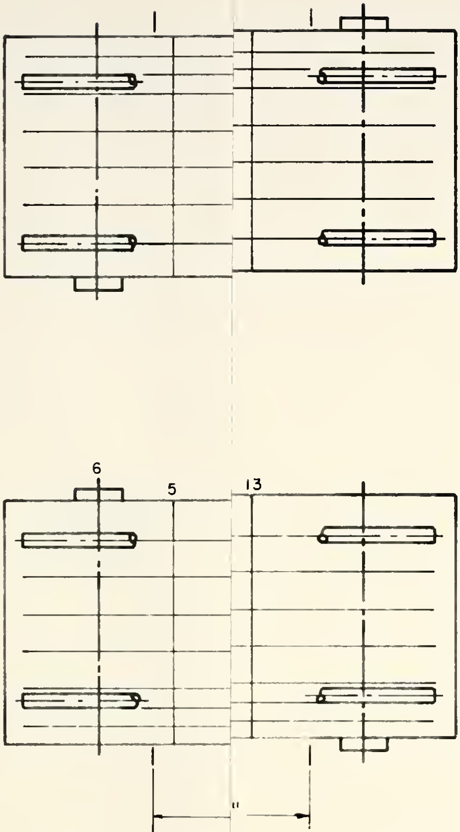
SR-4 Gage Locations:

0", 1" from bottom (S)

Cracks during first cycle
Cracks during repeated load
Cracks opening wide at failure
Strain gage

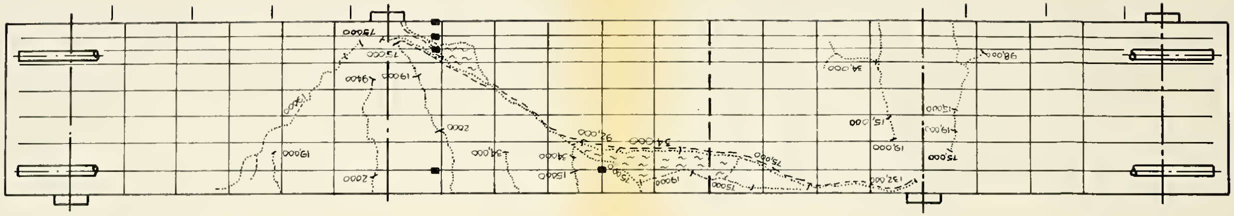
Scale: 1" = 8"

FIGURE 16. BEAM I BF-1

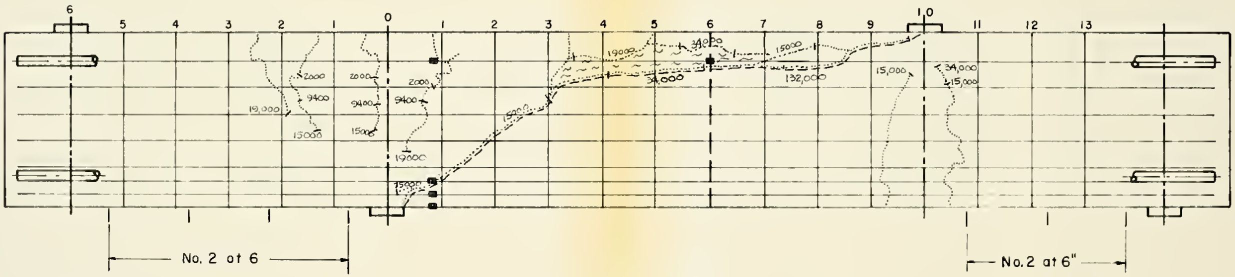


Foil Gage Location
No. 6 Bar -
" " -
" " -
" " -
-

SR-4 Gage Locat
0", 1", 2" fro



NORTH SIDE



SOUTH SIDE

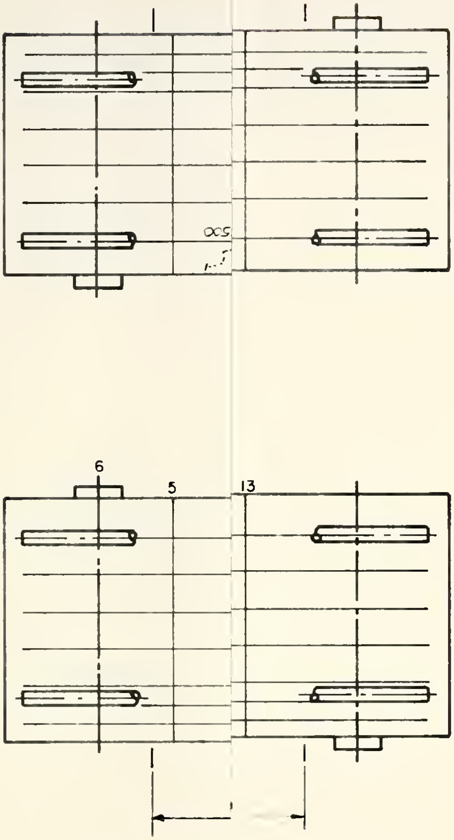
Foil Gage Locations:

No.6 Bar — 3 1/2" from support (N)
" " — " " " (S)
" " — 16" " " (N)
" " — 24" " " (S)

SR-4 Gage Locations:

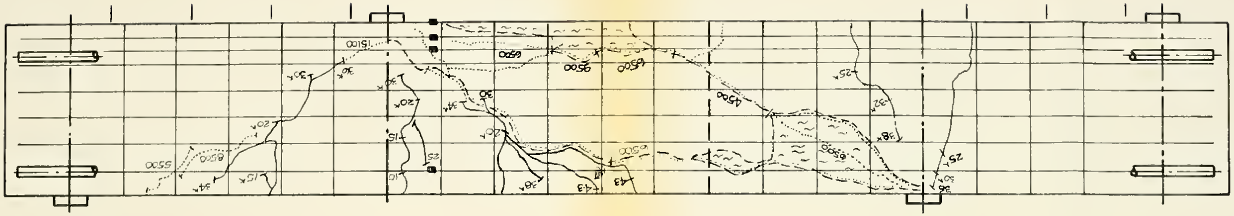
0", 1", 2" from bottom (N & S)

FIGURE 17. BEAM IBF-2

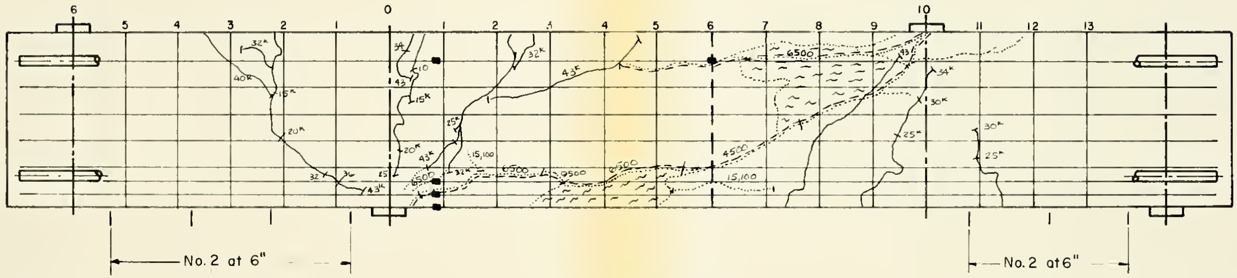


Foil Gage Location
No.6 Bar –
" " –
" " –
" " –

SR-4 Gage Locat
0", 1", 2" fro



NORTH SIDE



SOUTH SIDE

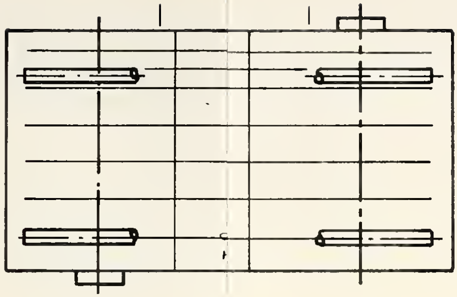
Foil Gage Locations:

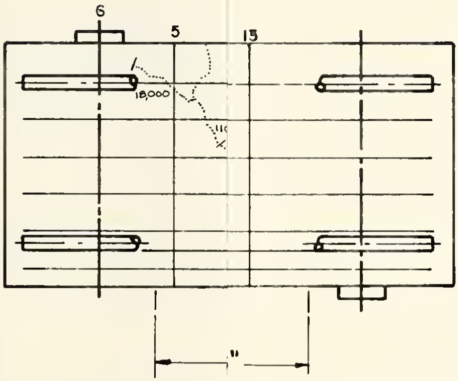
No.6 Bar — 3 1/2" from support (N)
" " — " " " (S)
" " — 16" " " (N)
" " — 24" " " (S)

SR-4 Gage Locations:

0", 1", 2" from bottom (N & S)

FIGURE 18. BEAM I BF-3





Foil Gage Locatio

No.6 Bar -

" " -

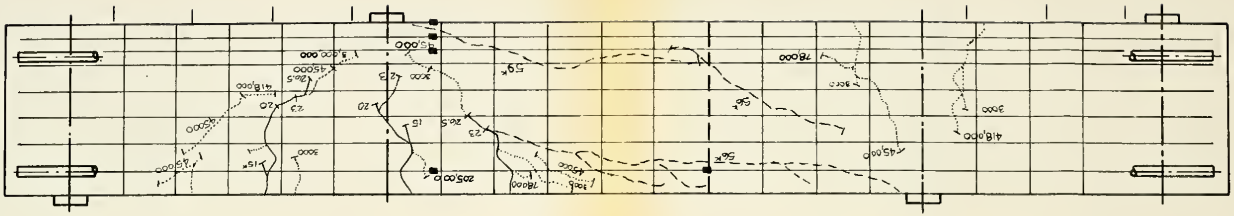
" " -

" " -

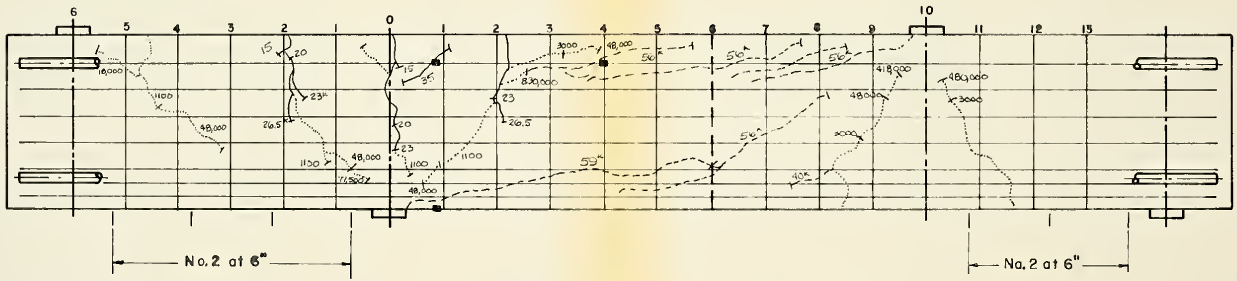
SR - 4 Gage Loca

0", 1", 2" fro

0" "



NORTH SIDE



SOUTH SIDE

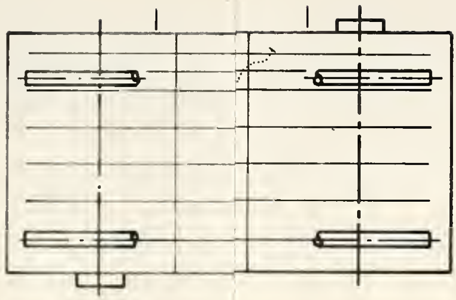
Foil Gage Locations:

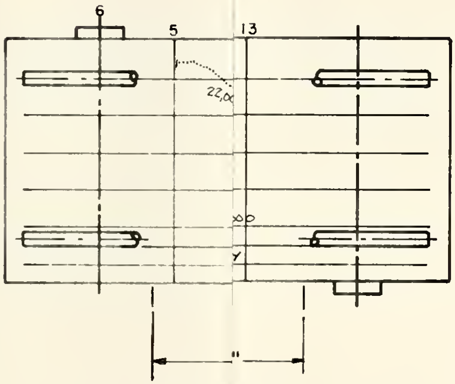
No.6 Bar	--	3 1/2"	from	support	(N)
"	"	—	"	"	"	(S)
"	"	—	16"	"	"	(S)
"	"	--	24"	"	"	(N)

SR - 4 Gage Locations:

0", 1", 2" from bottom (N)
0" " " (S)

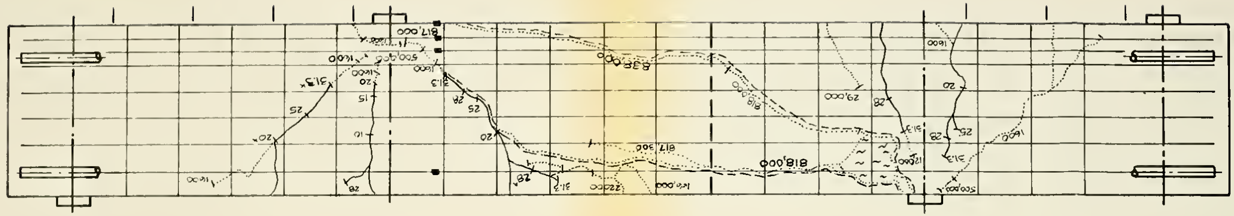
FIGURE 19. BEAM I BF-4



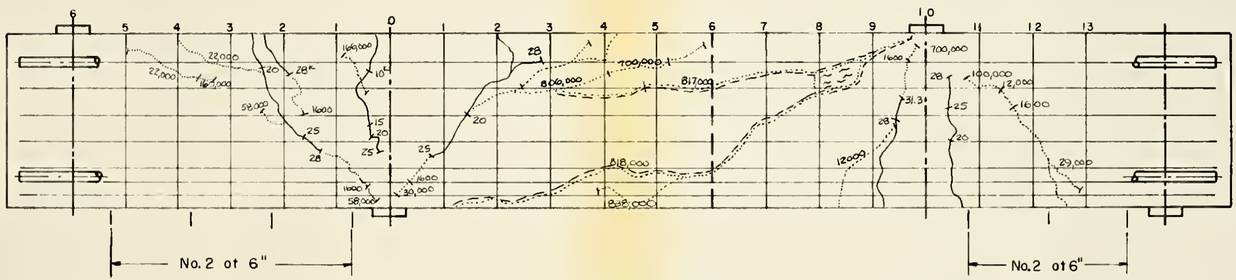


Foil Gage Location
No.6 Bar –

SR-4 Gage Loca
0", 1", 2" fro



NORTH SIDE



SOUTH SIDE

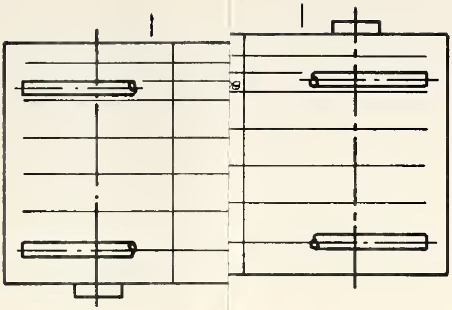
Foil Gage Locations:

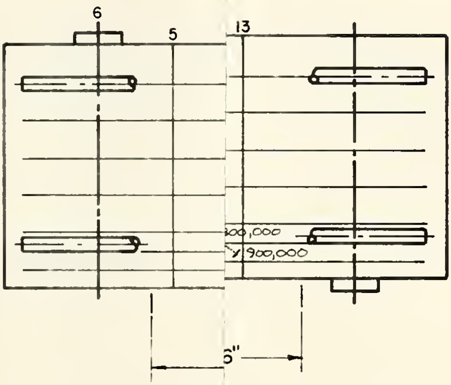
No. 6 Bar — 3 1/2" from support (N)

SR-4 Gage Locations:

0", 1", 2" from bottom (N)

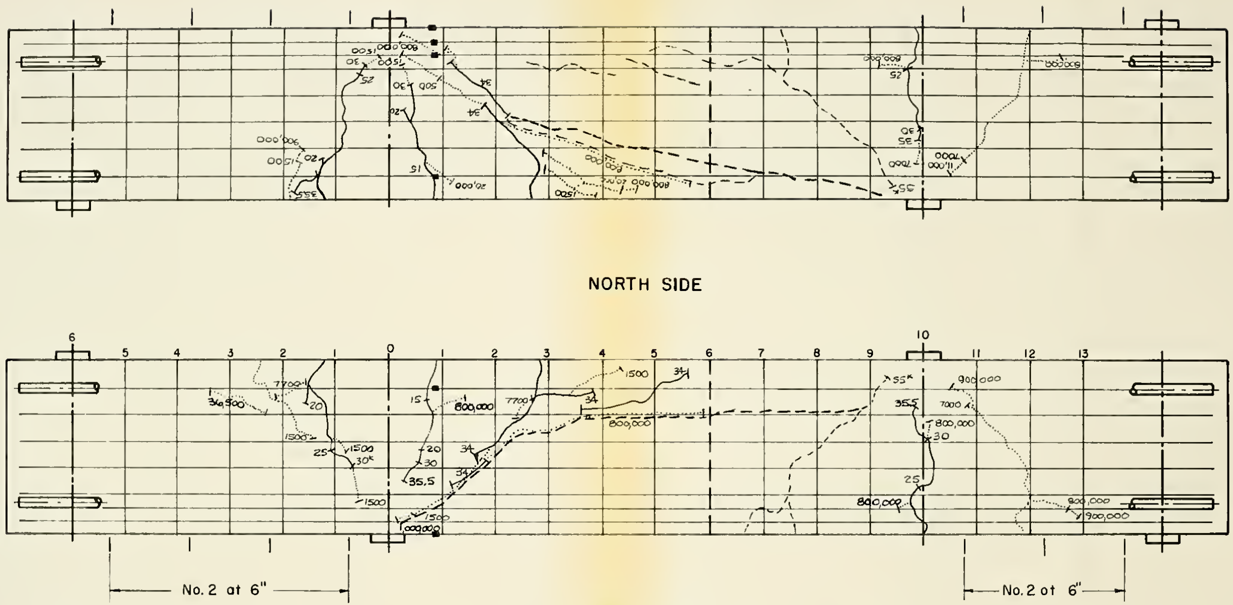
FIGURE 20. BEAM I BF-5





Fail Gage Location
No.6 Bar -
" " -

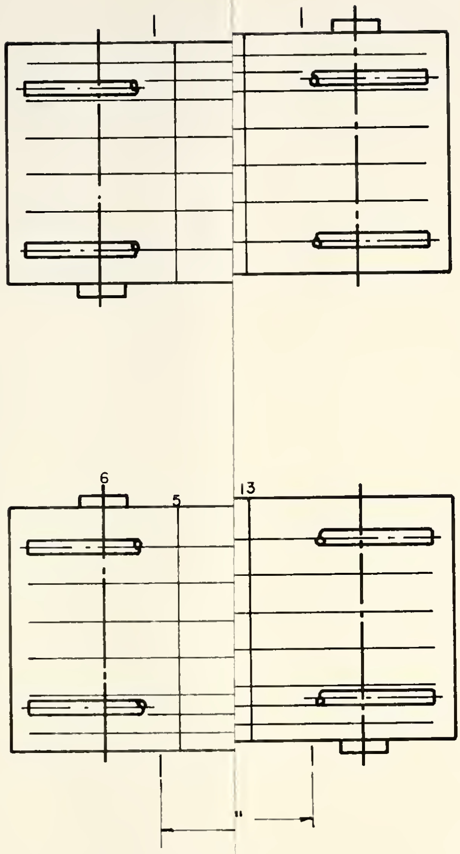
SR-4 Gage Loca
0", 1", 2" fr
0"



Foil Gage Locations:
No. 6 Bar — 3 1/2" from support (N)
" " — " " " (S)

SR-4 Gage Locations:
0", 1", 2" from bottom (N)
0" " " (S)

FIGURE 21 BEAM I BF-6



Foil Gage Location

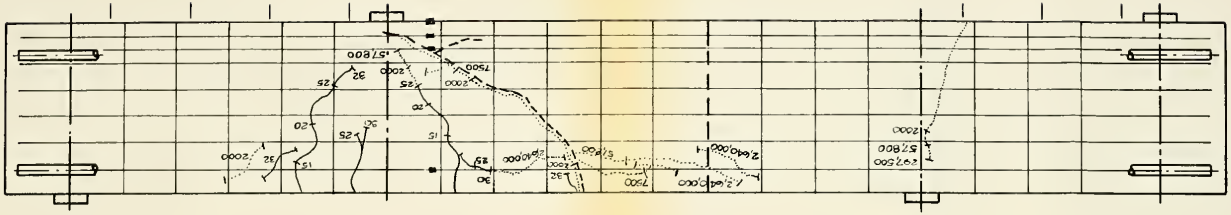
No. 6 Bar -

" " -

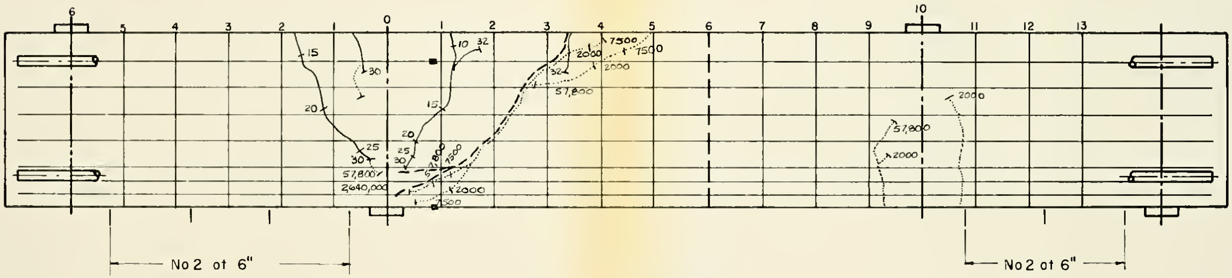
SR - 4 Gage Loc

0", 1", 2" fro

0" "



NORTH SIDE



SOUTH SIDE

Foil Gage Locations:

No. 6 Bar — 3 1/2" from support (N)
" " — " " " (S)

SR – 4 Gage Locations:

0", 1", 2" from bottom (N)
0" " " (S)

FIGURE 22 BEAM IBF-7

Series II BF

The specimens of this series were 8'-4" in length and had a shear span-to-depth ratio (a/d) of 2.88. The ratio of the restraining moment to positive moment (M_1/M_2) was 2.0. The major variable for this series was the maximum amplitude of the repeated load. The load-strain relationships are presented in Figures 23 and 24.

Beam II BF-1 (70% Ultimate)

The first flexural crack appeared at a load of 10^k. At a load of 30^k, there were four flexural cracks over the support and two flexural cracks under the interior load point. A diagonal crack formed and extended from within 3 inches of the support to the middle of the shear span on the north side of the member. As the load was increased to 33^k the diagonal crack appeared on the south side. The steel strain showed very little change from its value of 1174 MII (33,000 psi). The concrete strain practically doubled from 380 MII to 724 MII as the load was increased from 25^k to 30^k. Upon application of the repeated load there was considerable splitting along the longitudinal reinforcement with failure occurring after only 1500 cycles by enlargement of the diagonal crack.

Beam II BF-2 (60% Ultimate)

The crack scheme after the first cycle was almost identical to that of Beam II BF-1. After 2200 repetitions two parallel diagonal cracks appeared in the center of the shear span. The diagonal crack which was farther into the

shear span lengthened to the support and along the longitudinal steel to the interior load point to produce failure at 8800 cycles.

Beam II BF-3 (50% Ultimate)

The original cracking pattern was similar to the two previous specimens with flexural crack penetration to within 4 inches of the compressive face over the support after the first cycle load had reached 25^k. There was also a small flexural crack at 15 inches into the shear span. At 2500 cycles this crack had inclined and lengthened to within 4 inches of the compressive face. After 60,000 cycles the crack was 1 1/2 inches from the support. The strain level in the steel reached a maximum of 932 $\pm$ 400 MII (25,500 $\pm$ 10,000 psi) at 463,000 cycles before failure of the strain gages. The strain in the extreme surface of the concrete achieved a maximum average value of 750 $\pm$ 250 MII around one million cycles and then decreased gradually to an average level of 540 $\pm$ 220 MII at three million cycles. The diagonal crack extended 22 inches from the support when the repeated load was removed after three million cycles. During the static loading the diagonal crack increased in width when the load was increased to 30^k. The load was increased to 44^k, and the diagonal crack opened suddenly producing failure.

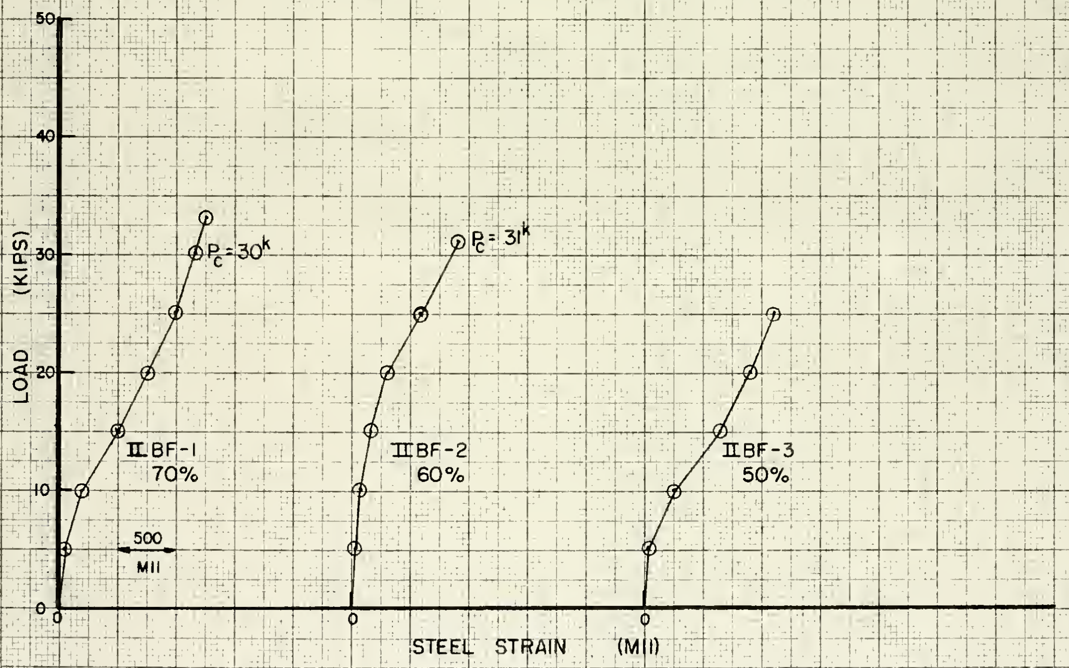


FIGURE 23. LOAD vs. STEEL STRAIN - SERIES II BF - FIRST CYCLE

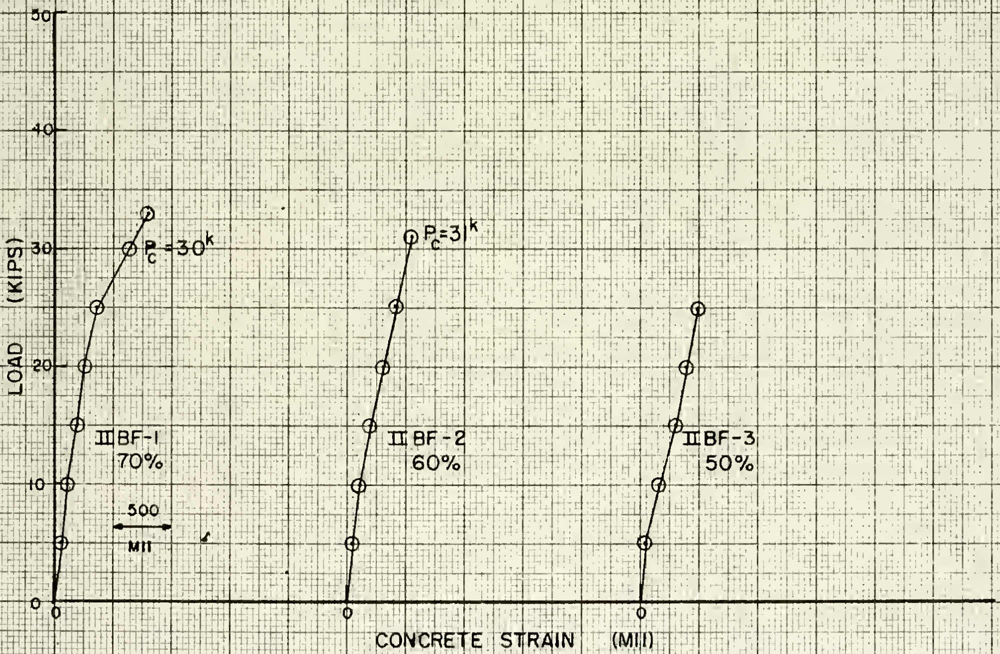


FIGURE 24. LOAD vs. CONCRETE STRAIN - SERIES II BF - FIRST CYCLE

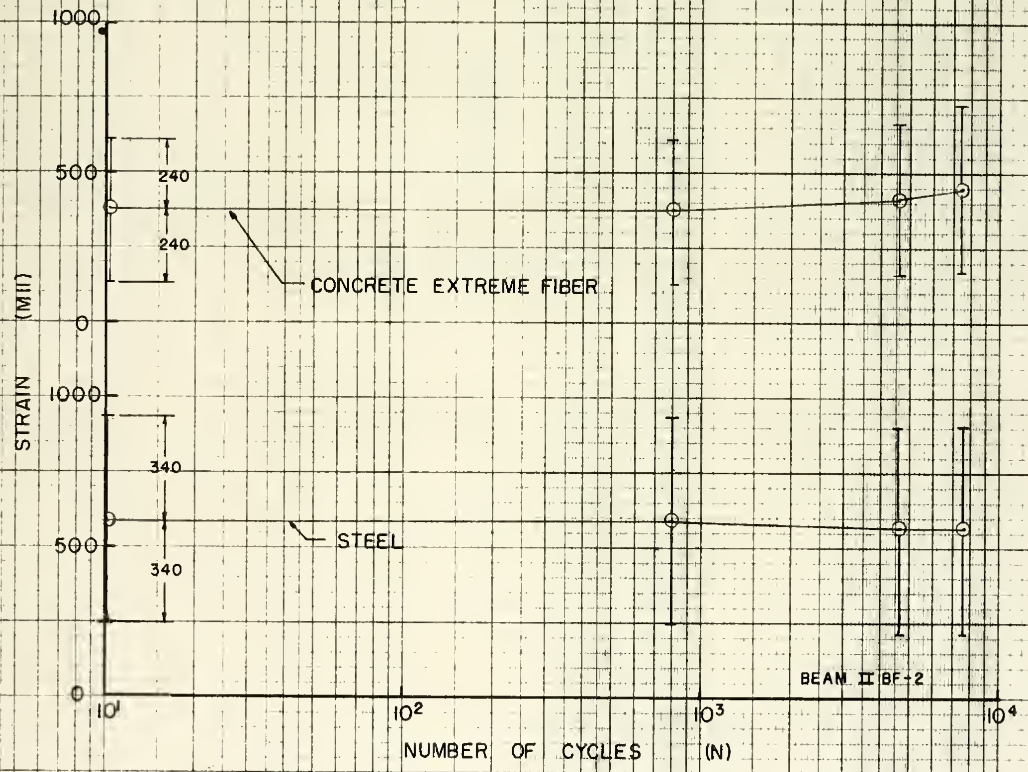
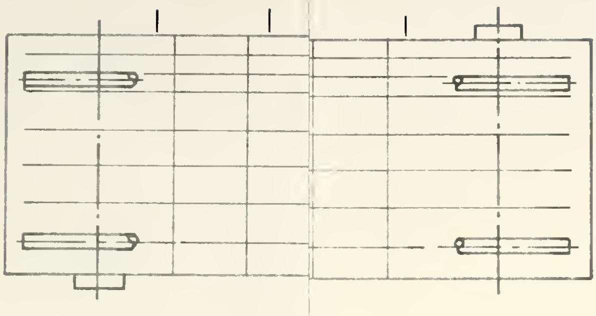
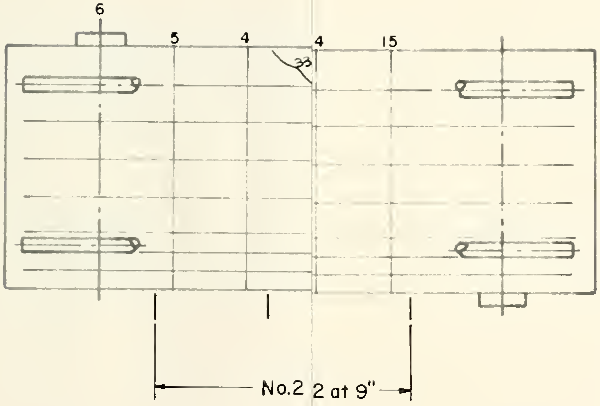


FIGURE 25. TYPICAL STRAIN vs. N RELATIONSHIP – SERIES II BF

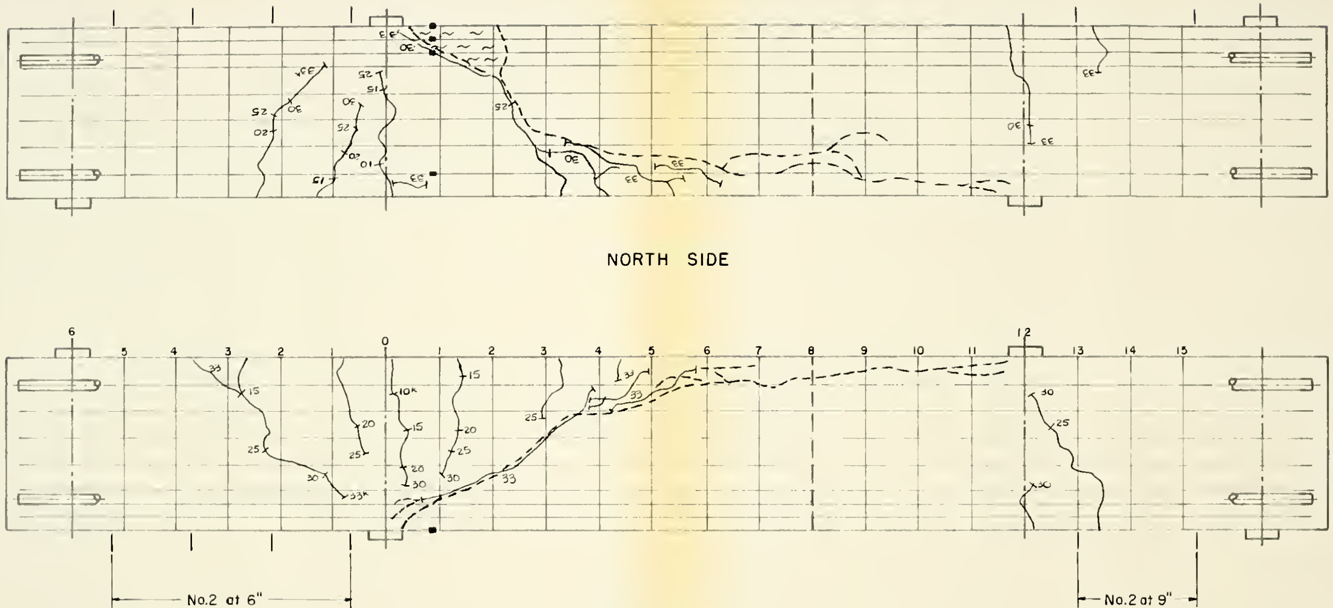




Foil Gage Locations:
No.6 Bar — 3 1/ ad

lure

SR-4 Gage Locations:
0", 1", 2" from bott
0" " "



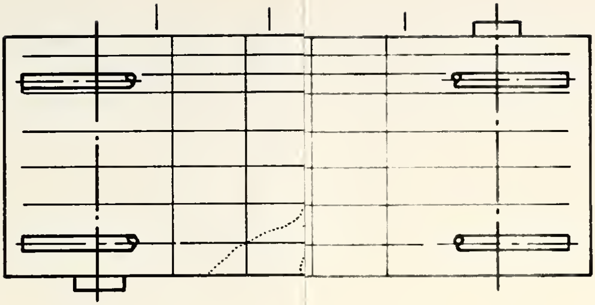
Foil Gage Locations:
No.6 Bar — 3 1/2" from support (N)

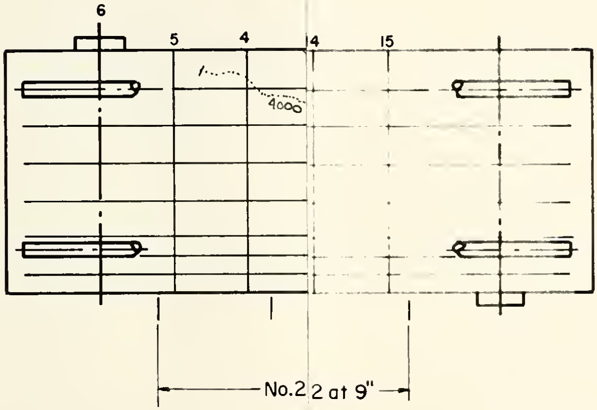
SR-4 Gage Locations:
0", 1", 2" from bottom (N)
0" " " (S)

25 Cracks during first cycle
6000 Cracks during repeated load
Cracks opening wide at failure
Strain gage

Scale: 1" = 8"

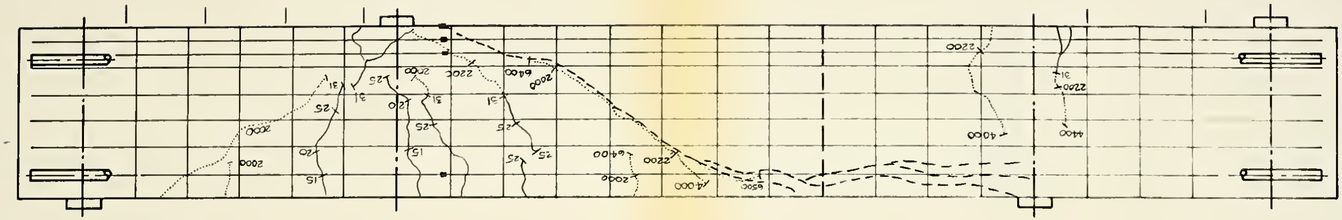
FIGURE 26. BEAM II BF-1



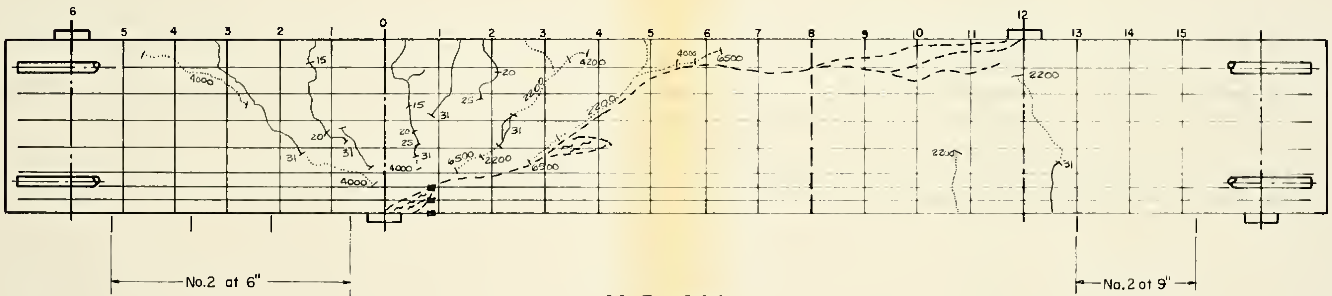


Foil Gage Locations:
No.6 Bar — 3 l/

SR-4 Gage Locations:
0", 1", 2" from bot



NORTH SIDE

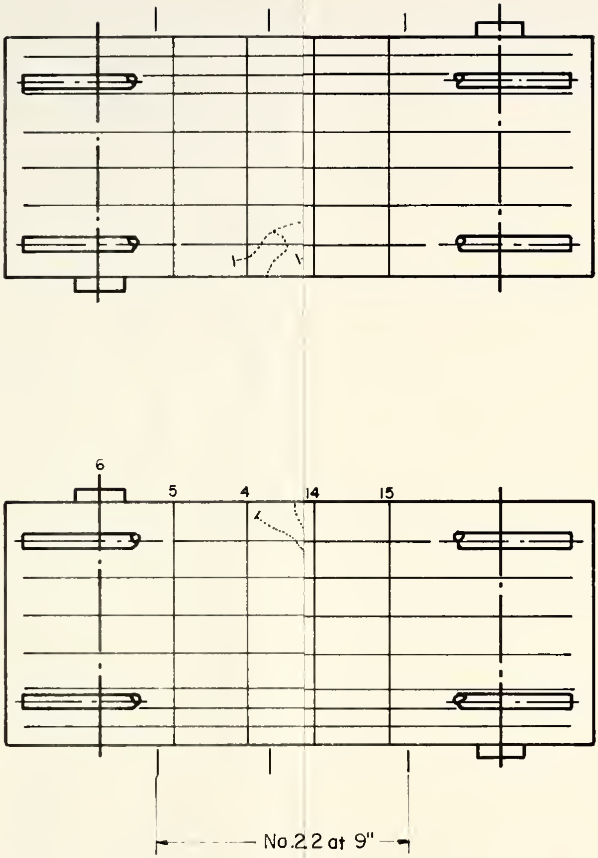


SOUTH SIDE

Foil Gage Locations:
No.6 Bar — 3 1/2" from support (N)

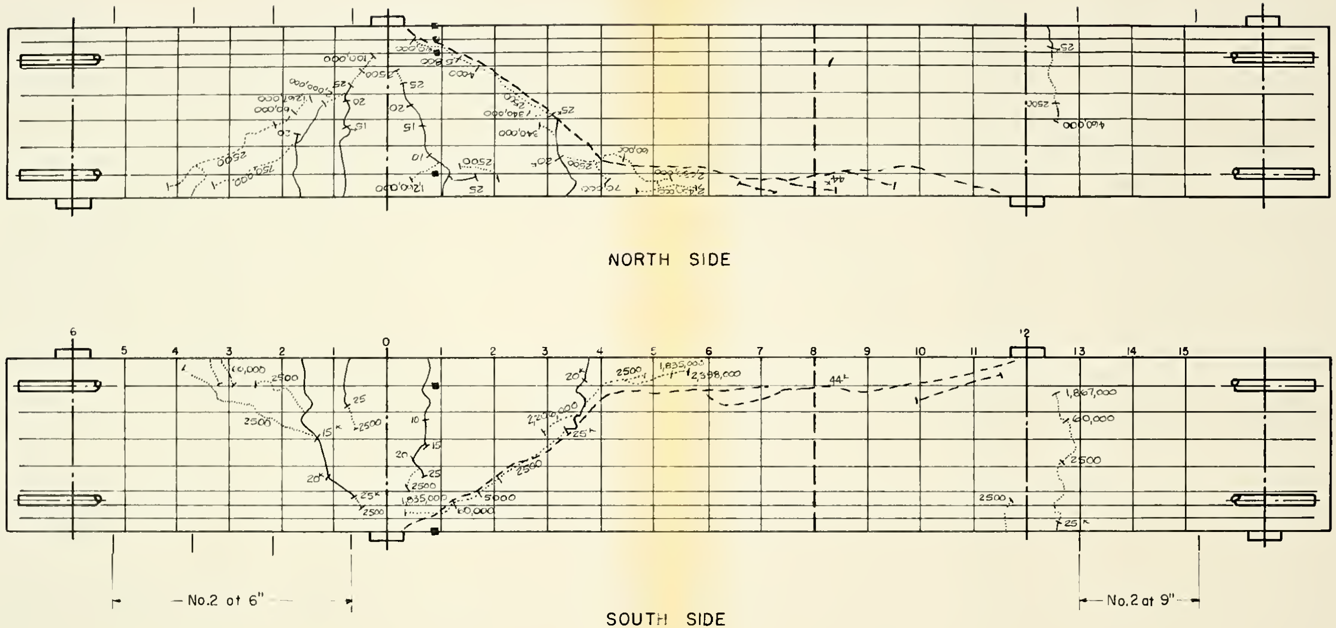
SR-4 Gage Locations:
0", 1", 2" from bottom (N & S)

FIGURE 27. BEAM II BF-2



Foil Gage Locations:
No. 6 Bars — 3 l

SR-4 Gage Locations:
0", 1", 2" from bo
0" "



Foil Gage Locations:

No. 6 Bars — 3 1/2" from support (N&S)

SR-4 Gage Locations:

0", 1", 2" from bottom (N)

0" " " (S)

FIGURE 28. BEAM II BF - 3

Series III BF

The shear span-to-depth ratio (a/d) for this series was 3.96. The length of the specimens was 9'-10", and the ratio of restraining moment to positive moment (M_1/M_2) was 2.0. The first cycle load-strain relationships are shown in Figures 29 and 30. Three specimens were tested at 60% of their predicted ultimate strengths.

Beam III BF-1 (70% Ultimate)

There were five flexural cracks as the static load reached 31.5^k. The flexural crack in the middle of the shear span and inclined toward the support because of diagonal tension. A second diagonal crack opened farther into the shear span at 3300 cycles. The average concrete strain at the compressive face increased gradually from 455 $\pm$ 320 MII at the start of repeated loading to 550 $\pm$ 330 MII at one million cycles. Increases in the lengths of the flexural cracks were noted up to 1,140,000 cycles. The specimen failed after 1,290,600 repetitions by brittle fracture of the reinforcement at the location of a flexural crack.

Beam III BF-2 (60% Ultimate)

Four flexural cracks had formed over the support at a load of 20^k in a similar manner to Beam III BF-1. At $P = 26.5^k$ on the first loading the maximum penetration was to within 4 inches from the support, and the flexural crack in the overhang headed toward the support. The diagonal crack in the shear span had lengthened slightly after 1000

cycles and was accompanied by several additional flexural cracks. One of the longitudinal bars indicated a progressive increase in strain toward yield from the beginning of the repeated load. At 3,773,100 repetitions the diagonal crack continued toward the support, and splitting along the tension steel was observed. The beam failed after 3,864,000 repetitions by brittle fracture of both bars near a flexural crack.

Beam III BF-3 (80% Ultimate)

The specimen cracked during the first cycle in a similar manner to the first two beams of Series III BF. At 800 cycles a diagonal crack opened in the shear span. The strain levels in the steel and concrete showed a small change (100 MII) during the repeated load. The repeated load was removed after 88,000 cycles because the specimen became increasingly unstable under loading. The static test resulted in a widening of the diagonal crack at $P = 40^k$ and failure at 48^k with the diagonal crack splitting open along the tension steel.

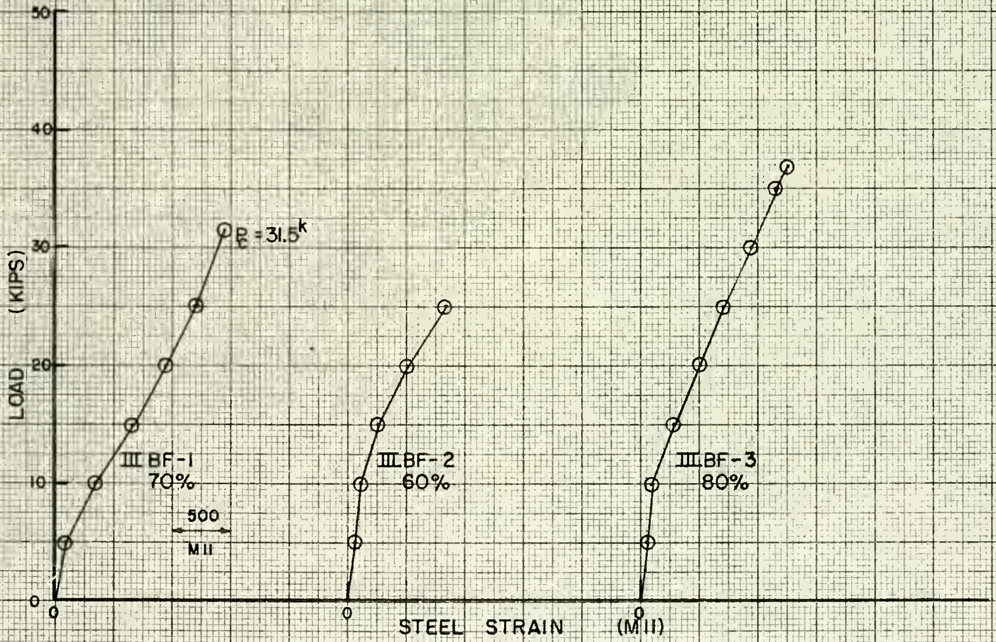


FIGURE 29. LOAD vs. STEEL STRAIN - SERIES III BF - FIRST CYCLE

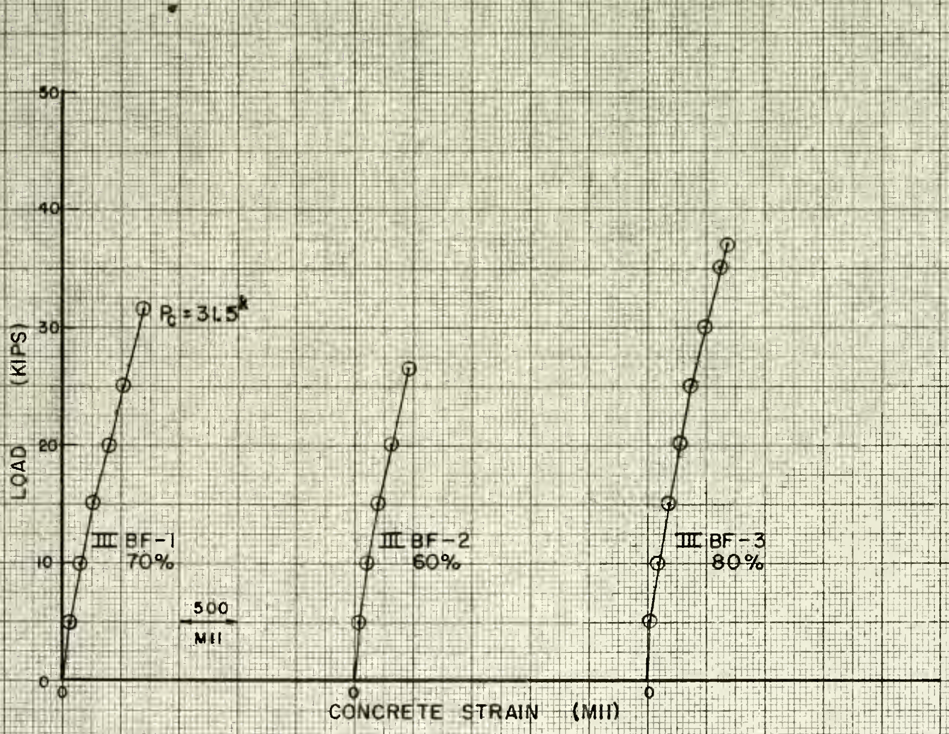


FIGURE 30. LOAD vs. CONCRETE STRAIN - SERIES III BF - FIRST CYCLE

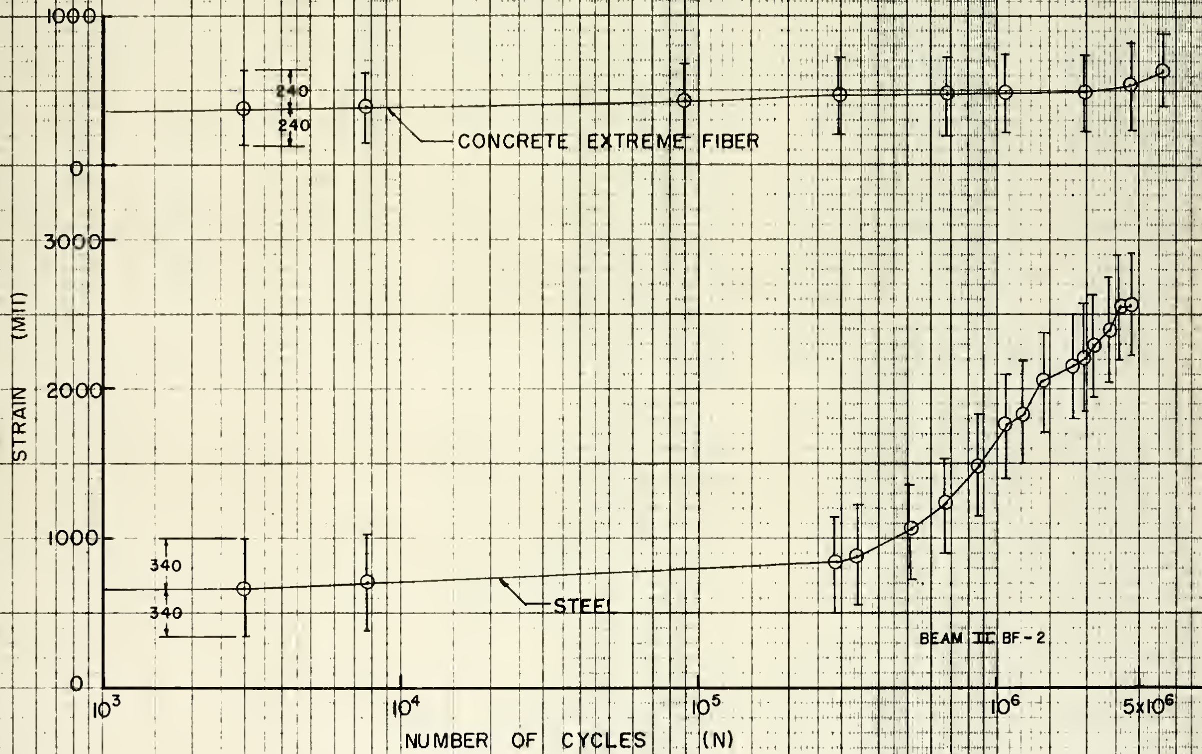
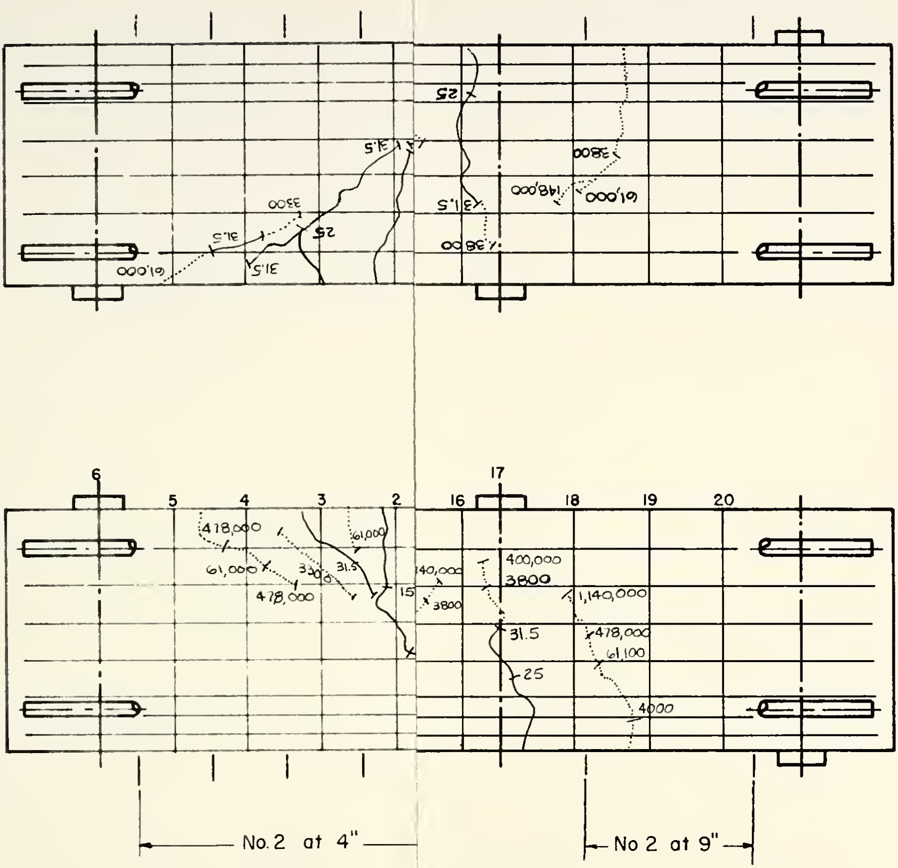


FIGURE 31. TYPICAL STRAIN vs. N RELATIONSHIP — SERIES III BF



Foil Gage Locations:

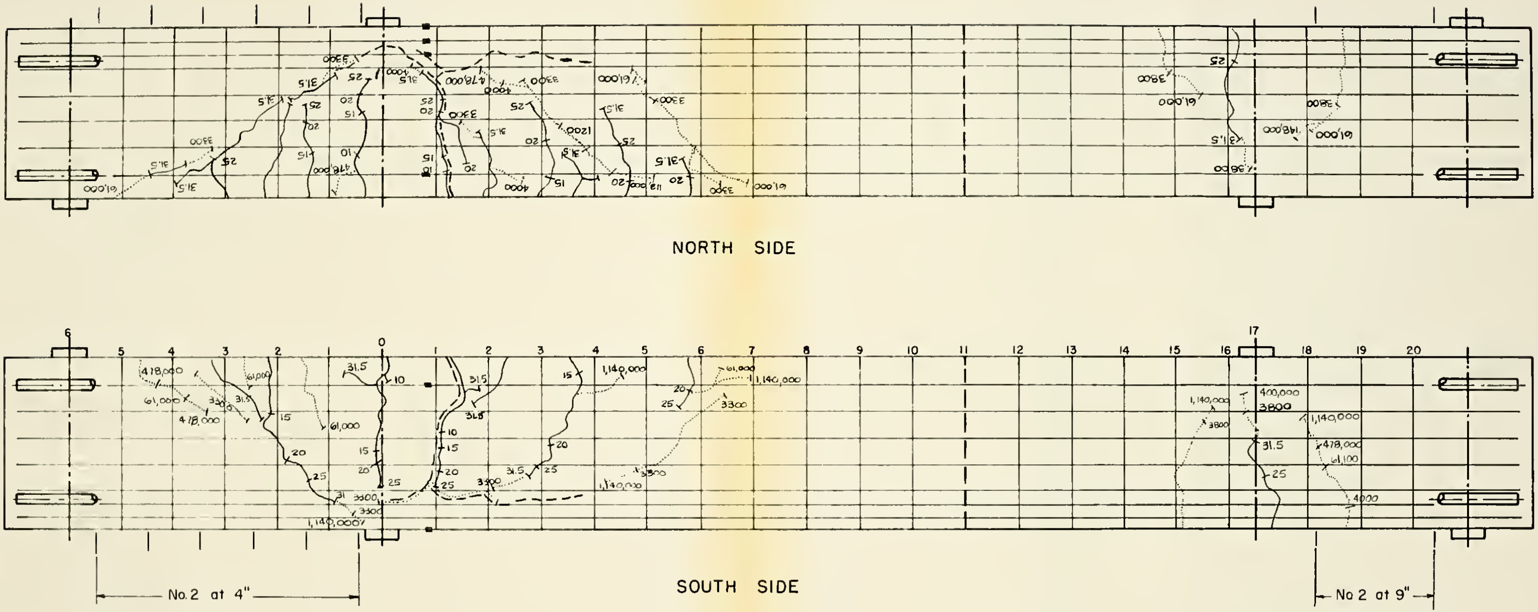
No.6 Bars — 3 1/

ure

SR-4 Gage Locations:

0", 1", 2" from botto

0" " "



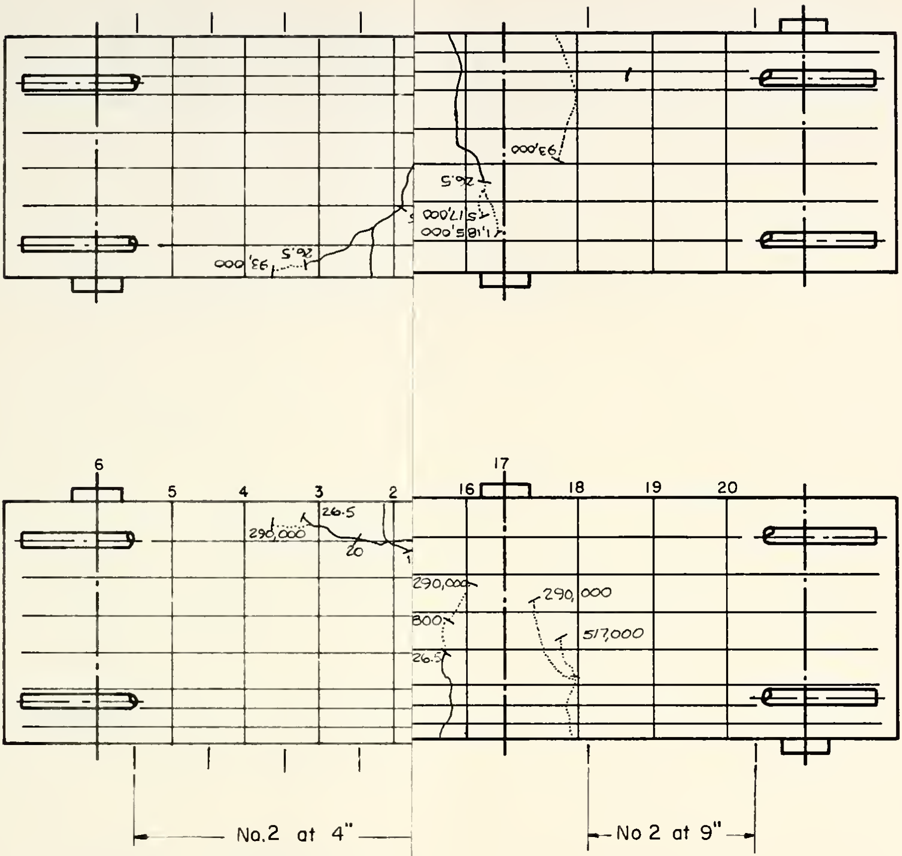
Foil Gage Locations:
No. 6 Bars — 3 1/2" from support (N & S)

SR-4 Gage Locations:
0", 1", 2" from bottom (N)
0" " " (S)

Cracks during first cycle
Cracks during repeated load
Cracks opening wide at failure
Strain gage

Scale: 1" = 8"

FIGURE 32 BEAM III BF-1



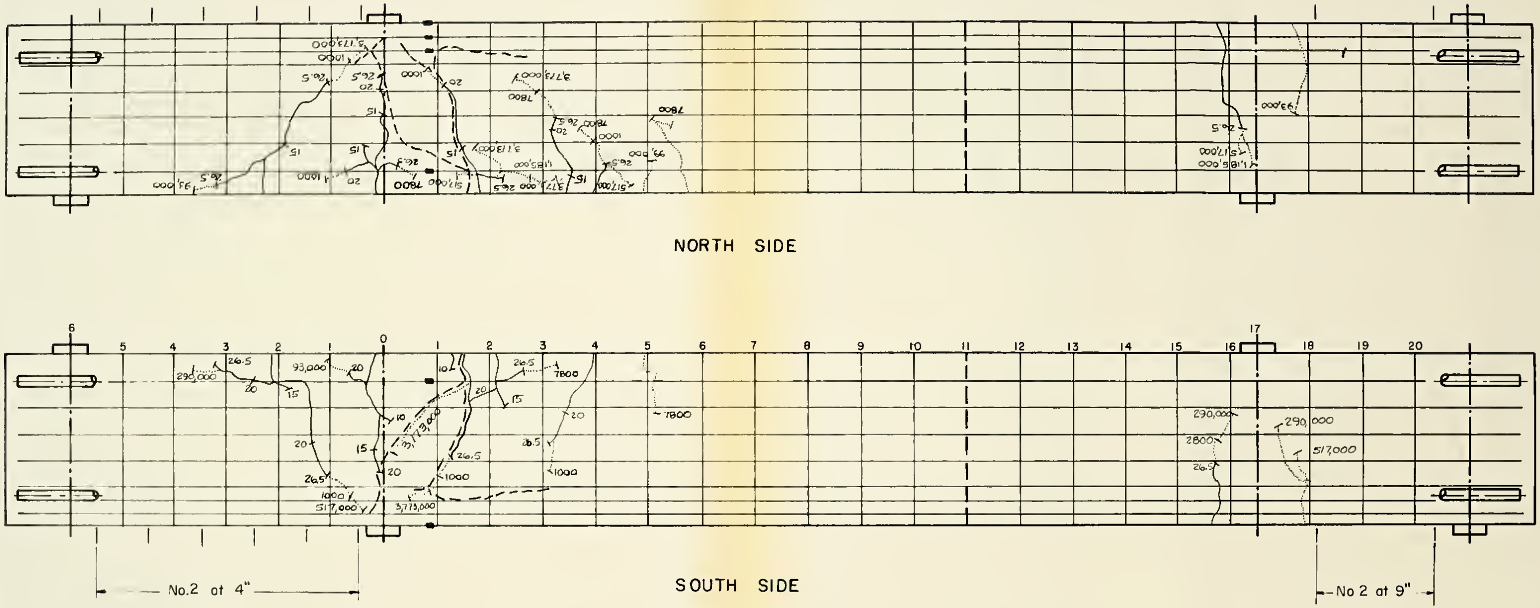
Foil Gage Locations:

No. Bars — 3 1/

SR-4 Gage Locations:

0", 1", 2" from bottor

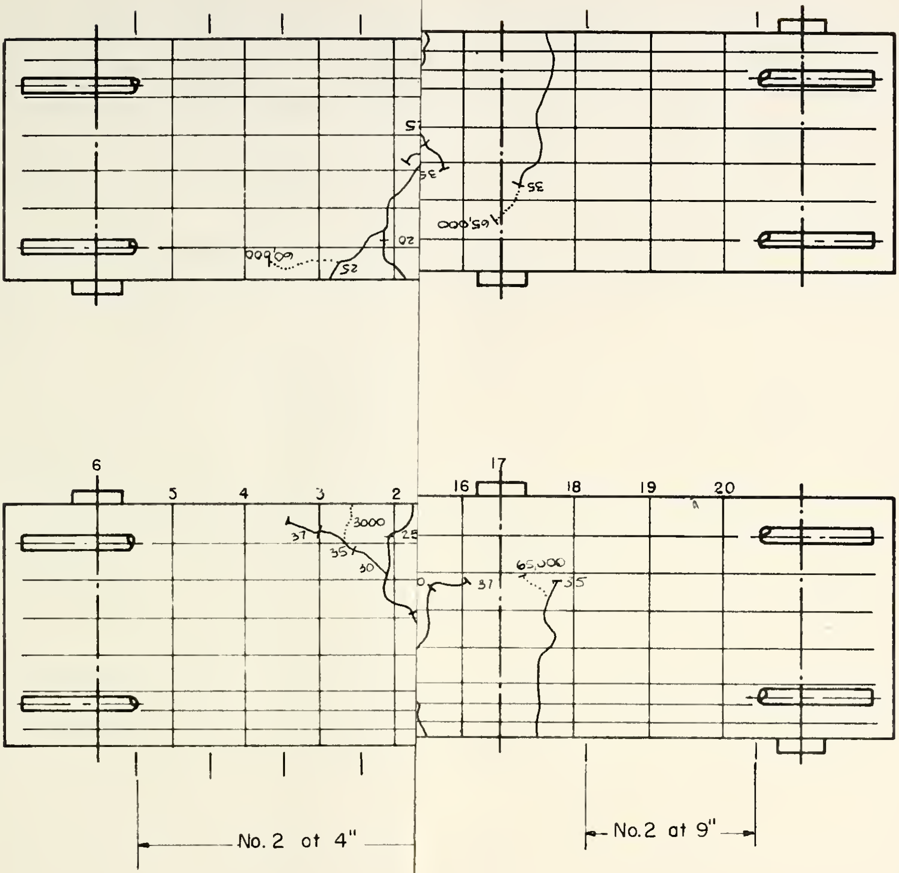
0" " "



Foil Gage Locations:
No. Bars — 3 1/2" from support (N&S)

SR-4 Gage Locations:
0", 1", 2" from bottom (N)
0" " " (S)

FIGURE 33. BEAM III BF-2



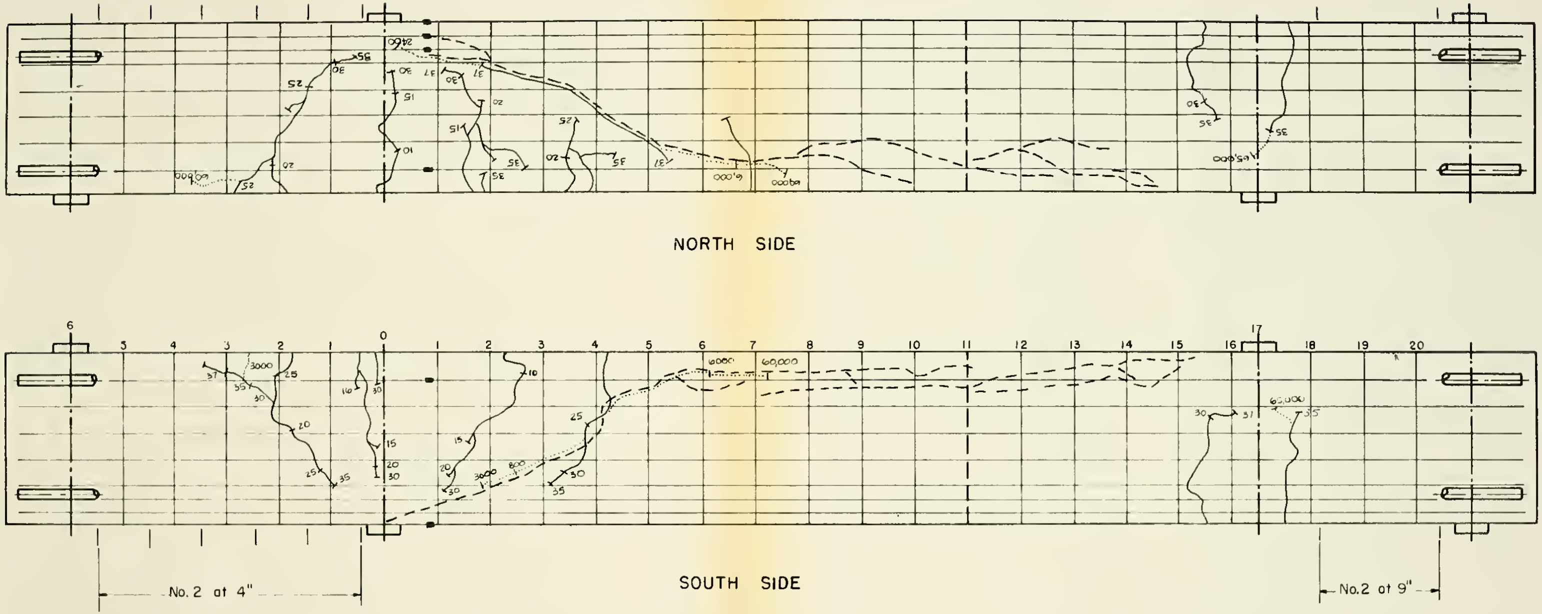
Foil Gage Locations:

No. 6 Bars — 3 1/

SR-4 Gage Locations:

0", 1", 2" from bott

0" " "



Foil Gage Locations:

No. 6 Bars — 3 1/2" from support (N & S)

SR-4 Gage Locations:

0", 1", 2" from bottom (N)

0" " " (S)

FIGURE 34. BEAM III BF-3

Series II BFR

The series is similar to Series IIBF with the exception that web reinforcement was provided in the critical shear span. The shear span-to-depth ratio (a/d) was 2.88 as in Series II BF. The details of the web reinforcement may be found in Table 3. The beams in this series did not show the same vulnerability to failure by diagonal tension as did the companion specimens of Series II BF.

Beam II BFR-1 (70% Ultimate)

The first flexural crack appeared at a load of 10^k. When the first cycle load reached 30^k, there were three flexural cracks over the support which had penetrated to 4 inches from the extreme surface. At $P = 30^k$ a diagonal crack formed in the overhang. The flexural crack in the middle of the shear span on the north side of the specimen developed into a diagonal crack at $P = 35^k$. An internal redistribution of stress was noted at $P = 30^k$ as the instrumented stirrup 9 inches from the centerline of the support (Stirrup a, Figure 38) began to indicate tensile strains. As the load was increased to 41.5^k, a second diagonal crack appeared at the middle of the shear span. The steel strain level was 1750 MII (51,000 psi) in the longitudinal bars; the maximum stirrup strain was 619 MII (16,500 psi); and the average concrete strain at the compressive face was 764 MII (3500 psi). After 3000 cycles the diagonal cracks had lengthened to 3 inches from the support. The instrumented stirrup nearest the support indicated yielding at 49,500

cycles. However, after 365,500 repetitions the behavior of the beam was unchanged. Failure occurred at 1,212,600 cycles with brittle fracture of the longitudinal bars 5 inches from the support centerline.

Beam II BFR-2 (60% Ultimate)

This specimen behaved similarly to Beam II BFR-1. The diagonal crack occurred at $P = 35^k$. The strain gages on the longitudinal reinforcement failed after 24,500 cycles. At 1,380,000 repetitions a second diagonal crack in the shear span opened 28 inches from the support. At 1,570,000 cycles stirrup (b) (Figure 39) indicated yielding. The remaining instrumented stirrups were no longer operable. The beam failed after 1,782,000 cycles by brittle fracture of the longitudinal reinforcement in the same fashion as Beam II BFR-1.

Beam II BFR-3 (80% Ultimate)

There were two diagonal cracks in the specimen at a load of $P = 37^k$. This was evident from the increasing strains in the stirrups. In addition there were characteristic flexural cracks over the support. After 40,000 cycles there was considerable enlargement of the diagonal cracks. The specimen failed by brittle fracture of the longitudinal steel after 296,400 cycles. A view of the fractured bars which was the mode of failure of Beams I BF-7, III BF-1 and 2, and all beams of Series II BFR may be seen in Figure 41, where the concrete has been removed to reveal the bars.

A summary of the fatigue lives of the specimens in relation to the level of the repeated load may be seen in Figure 42.

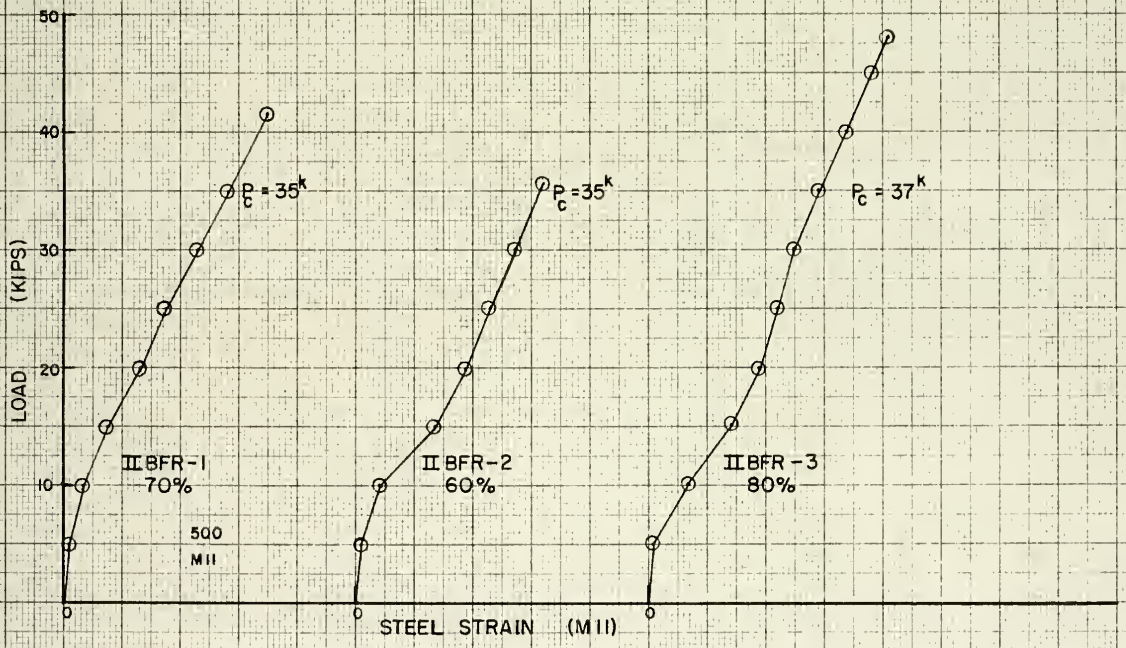


FIGURE 35. LOAD vs. STEEL STRAIN - SERIES II BFR - FIRST CYCLE

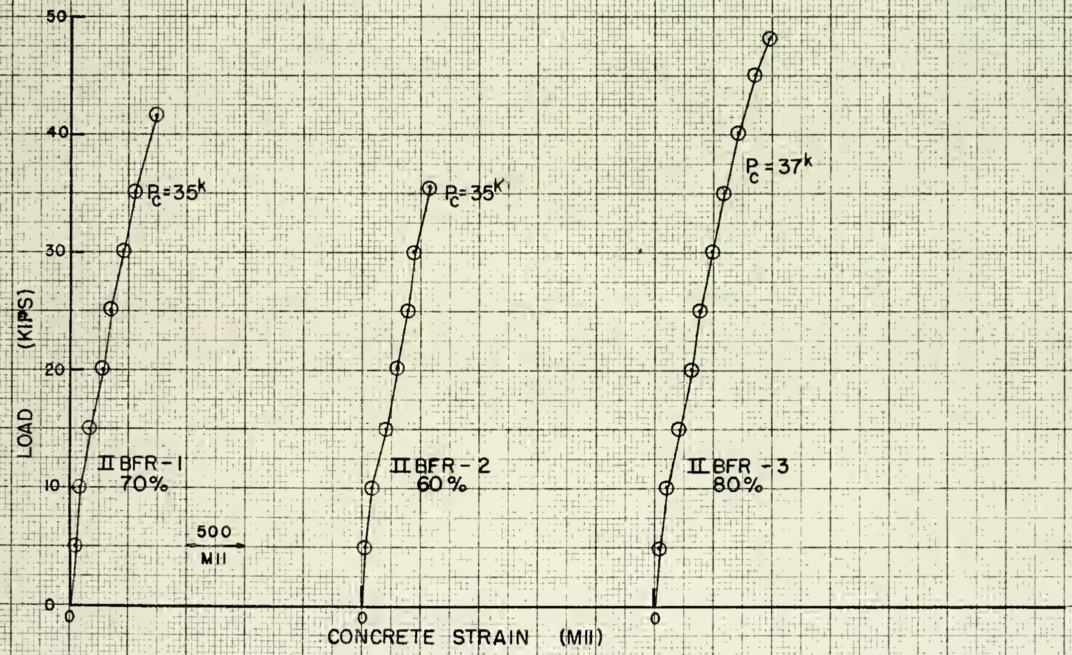


FIGURE 36. LOAD vs. CONCRETE STRAIN - SERIES II BFR - FIRST CYCLE

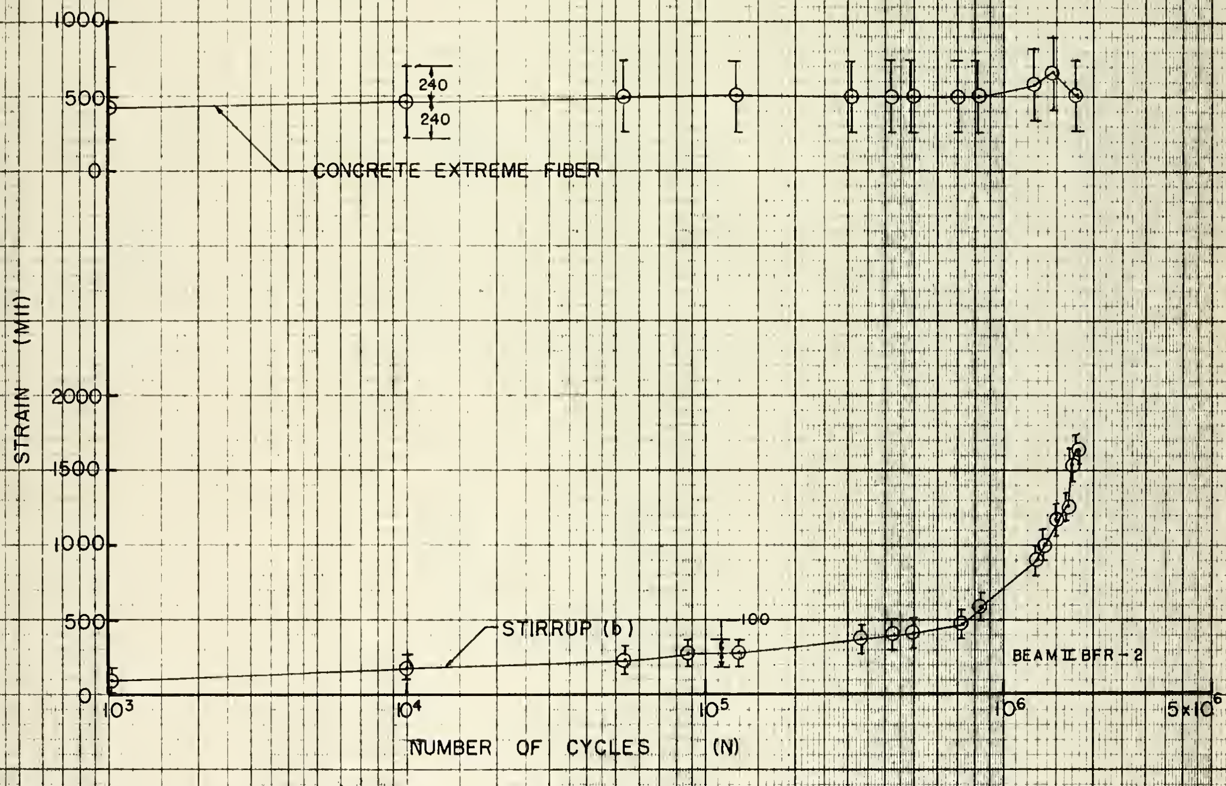
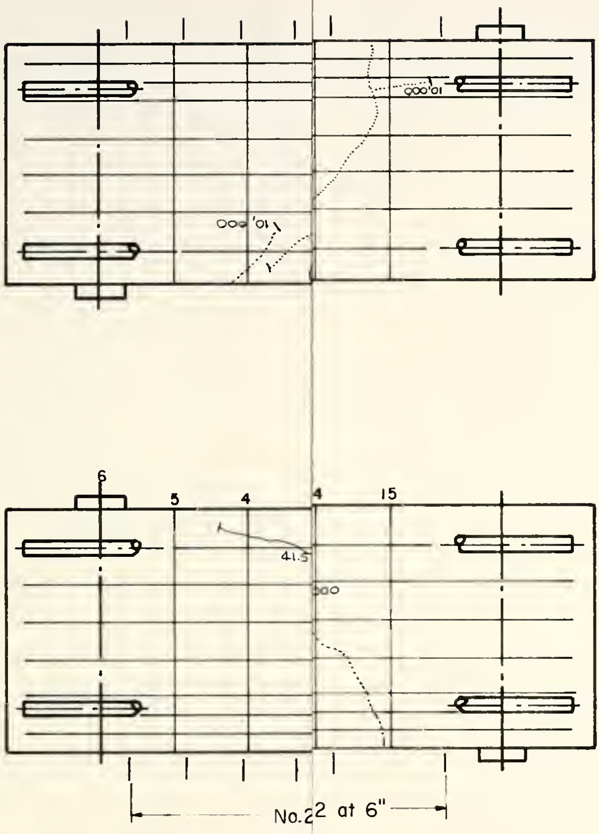


FIGURE 37 TYPICAL STRAIN vs. N RELATIONSHIP — SERIES II BFR

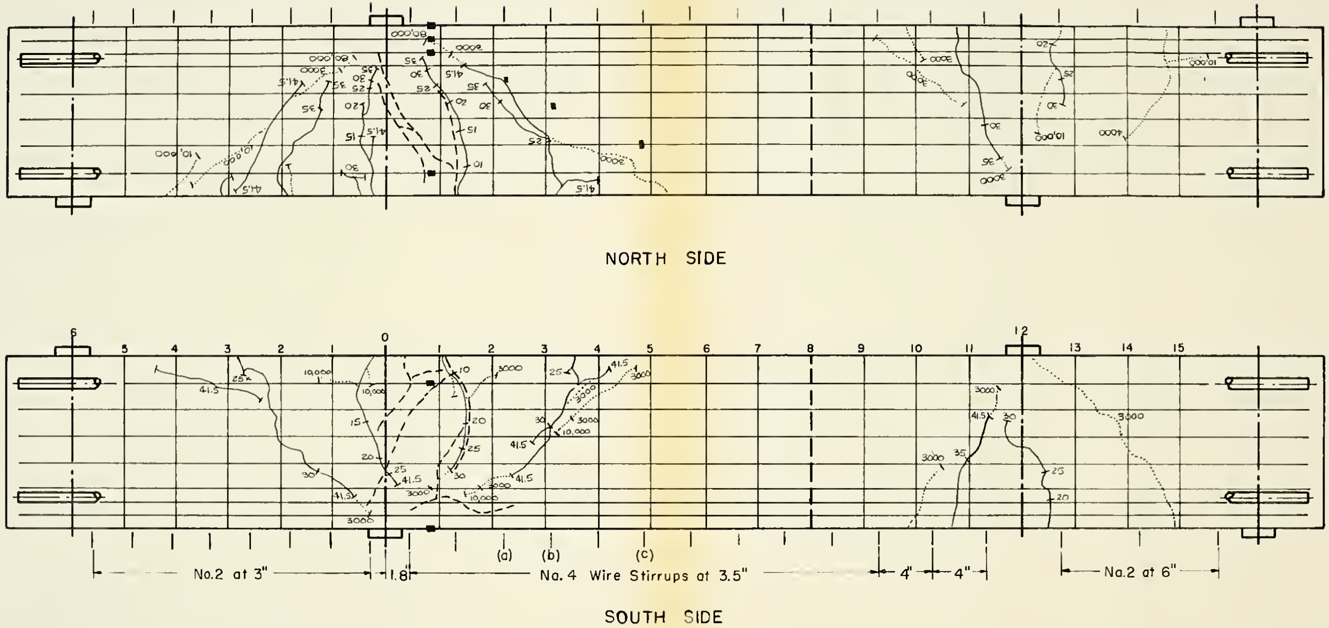


Foil Gage Locations:

No. 6 Bars — 3 1/ad
Stirrup (a) — 4"
" (b) — 6" lure
" (c) — 9"

SR-4 Gage Locations:

0", 1", 2" from bott
0" " "



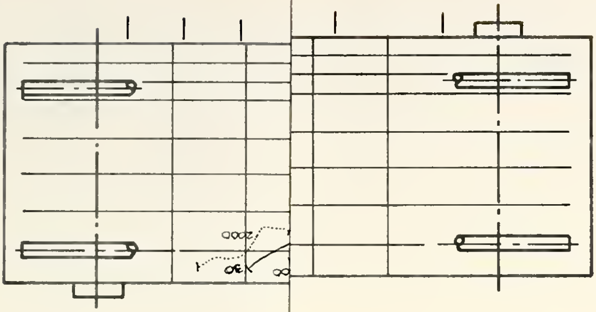
Foil Gage Locations:
No. 6 Bars — 3 1/2" from support (N & S)
Stirrup (a) — 4" " bottom (N)
" (b) — 6" " " "
" (c) — 9" " " "

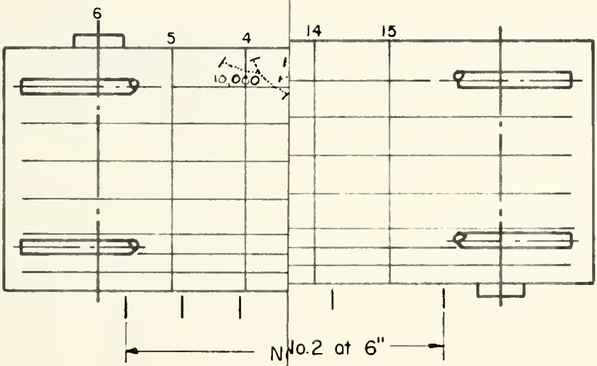
SR-4 Gage Locations:
0", 1", 2" from bottom (N)
0" " " (S)

Cracks during first cycle
Cracks during repeated load
Cracks opening wide at failure
Strain gage

Scale: 1" = 8"

FIGURE 38. BEAM II 8FR-1

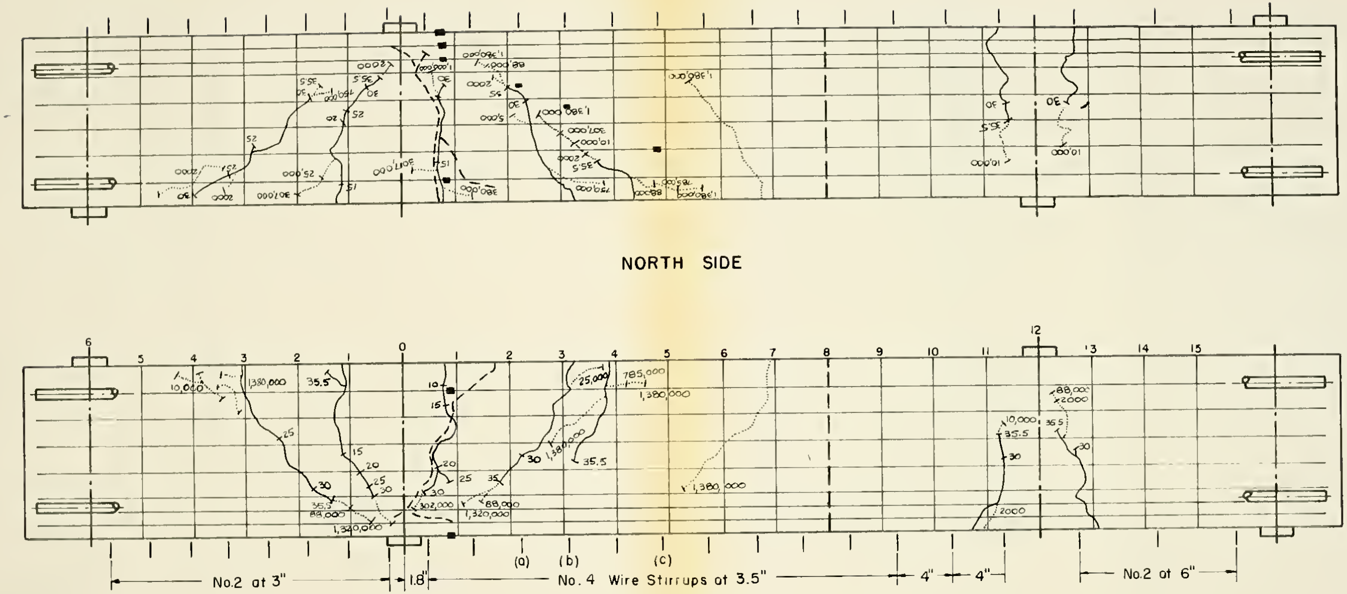




Foil Gage Locations:

No. 6 Bars — 3
Stirrup (a) — 4"
" (b) — 6"
" (c) — 9"

SR-4 Gage Locations:
0", 1", 2" from bo
0" "



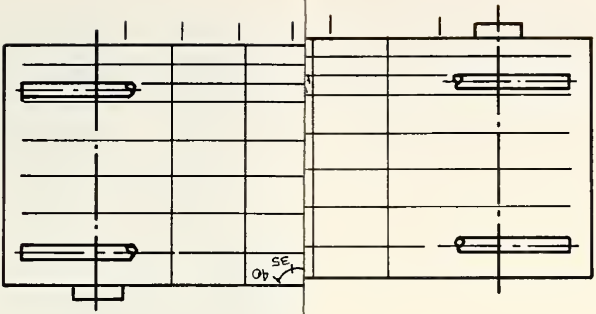
Foil Gage Locations:

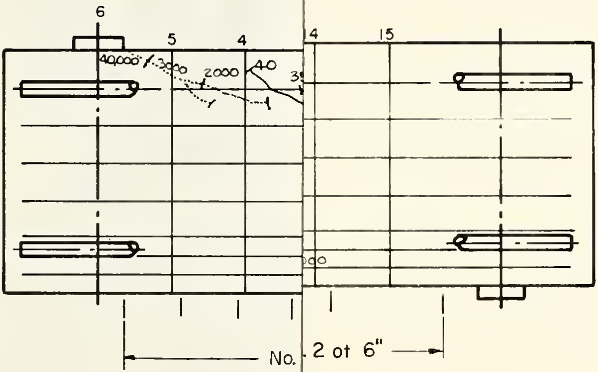
No.6 Bars — 3 1/2" from support (N&S)
Stirrup (a) — 4" " bottom (N)
" (b) — 6" " " "
" (c) — 9" " " "

SR-4 Gage Locations:

0", 1", 2" from bottom (N)
0" " " (S)

FIGURE 39. BEAM II BFR-2



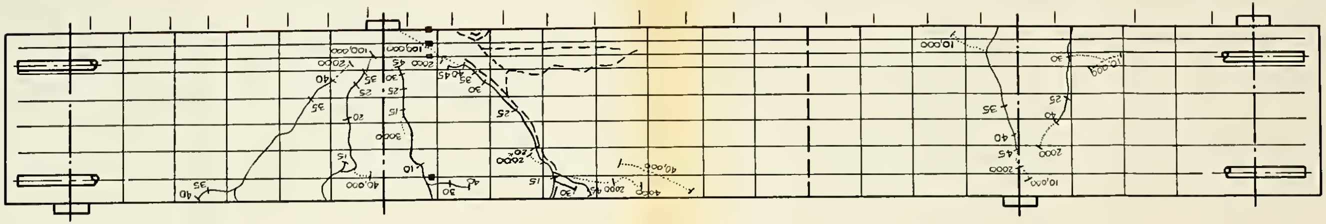


Foil Gage Locations:

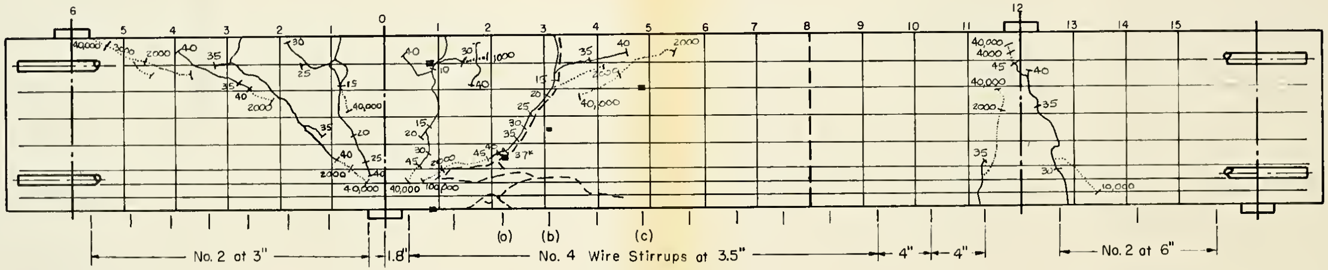
No.6 Bars — 3 l
Stirrup (a) — 4"
" (b) — 6"
" (c) — 9"

SR-4 Gage Locations:

0", 1", 2" from bott
0" " "



NORTH SIDE



SOUTH SIDE

Foil Gage Locations:

No.6 Bars — 3 1/2" from support (N & S)
Stirrup (o) — 4" " bottom (N)
" (b) — 6" " " "
" (c) — 9" " " "

SR-4 Gage Locations:

0", 1", 2" from bottom (N)
0" " " (S)

FIGURE 40. BEAM II BFR-3

FIGURE 41. A TYPICAL BRITTLE FRACTURE OF THE REINFORCEMENT

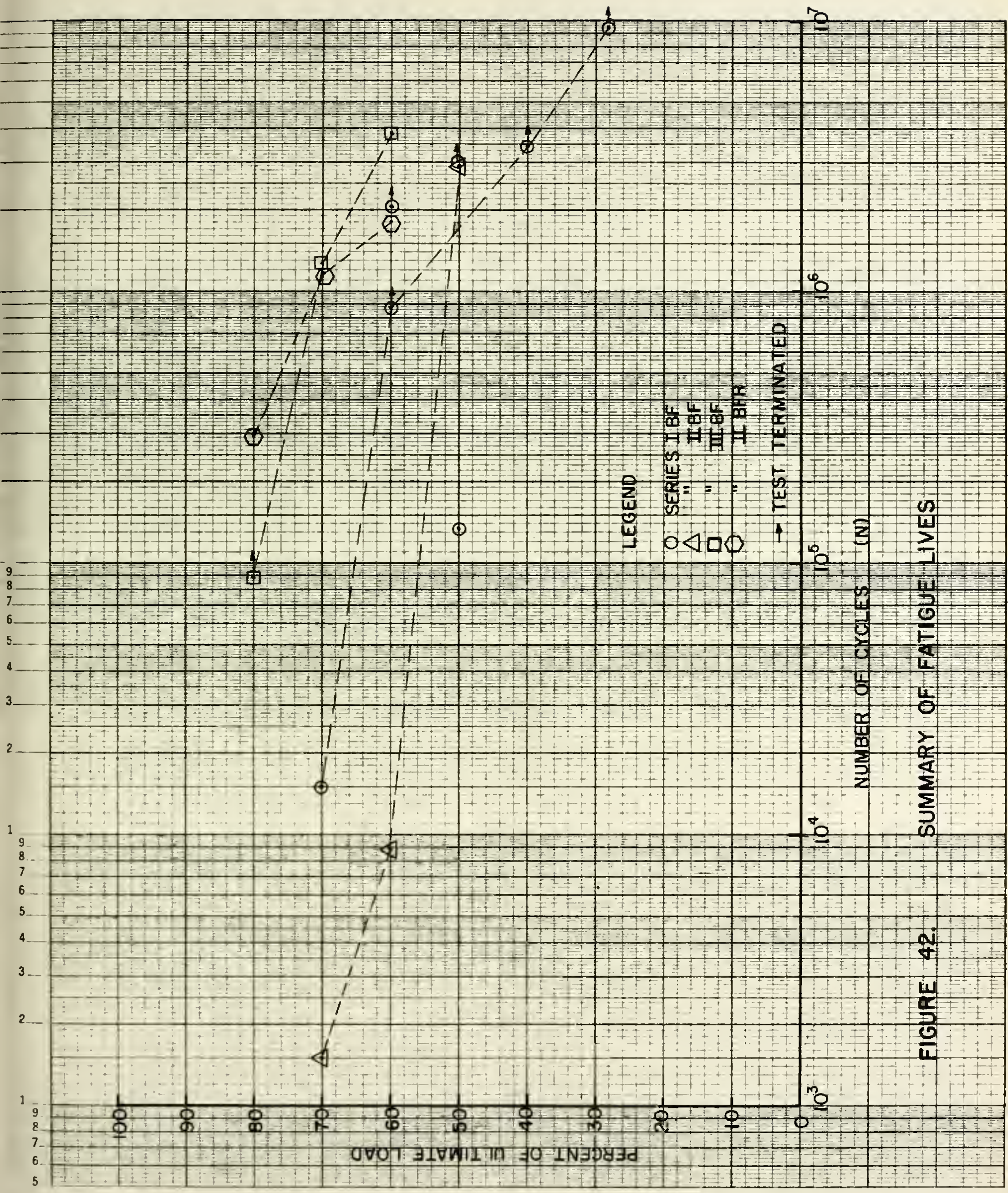


FIGURE 42. SUMMARY OF FATIGUE LIVES

DISCUSSION OF TEST RESULTS

Modes of Failure

The beams of Series I BF failed either by diagonal tension or shear-compression. However, there was no definite relationship to indicate whether the failure mode would be shear-compression or diagonal tension. With the exception of the two specimens tested at 28.5% and 40% of their predicted ultimate strengths, the diagonal cracking load for the beams of this series was fairly constant. The higher cracking load for the two exceptions is probably due to an increase in the concrete strength with time as they were tested statically after they had been loaded repeatedly. The fatigue life of a member increased with a reduction of the magnitude of the applied load. This behavior is in agreement with the results of other investigators[6,17,18,19]. The static mode of failure for the series as found in Harvey's tests[9] was shear-compression. In general, the results of the repeated load tests suggest a trend toward failure by diagonal tension for similar specimens.

Series II BF beams all failed by diagonal tension. Again, the diagonal cracking load was nearly the same for the series. The static mode of failure was shear-compression as indicated by Harvey[9]. Thus, there is also a change in the

failure mode for beams with a shear span-to-depth ratio of 2.88 when the loading is repeated.

The beams of Series III BF failed by fatigue of the longitudinal reinforcement although evidence of diagonal cracking was present. In comparison, the static mode of failure for beams with the same a/d ratio of 3.96 was diagonal tension. Once again, the cracking loads were consistent.

Series II BFR beams failed with fatigue of the longitudinal reinforcement as was found in Series III BF. These beams were similar to Beam II B-3 of Harvey's investigation[9] which failed by shear-compression. In comparison to beams of Series II BF which had no web reinforcement, there was a greater resitance to failure with the addition of a high percentage of stirrups.

Factors Affecting Beam Behavior

There are five major factors which affect the shear strength of reinforced concrete beams subjected to fatigue loading: concrete strength; percentage of longitudinal reinforcement; the amount of web reinforcement; the shear span-to-depth ratio; and the magnitude of the repeated load. It was intended that the first two parameters be held constant for this investigation. However, there was some unintentional variation in the concrete strengths. The shear span-to-depth ratio, the magnitude of the repeated loads, and the amount of web reinforcement were the major variables.

Shear Span-to-Depth Ratio

The summary of test results in Table 4 indicates a decrease in average shearing stress as the a/d ratio increases. The beams having similar concrete strengths and the same tension steel show the following:

Series	a/d	Avg.* v_c = V/bd (psi) (At diagonal ten. cracking)	Avg.* v_u = V/bd (At failure)
I BF	2.16	206	248
II BF	2.88	133	167
III BF	3.96	117	131
II BFR	2.88	158	193

*Avg. of all beams in each series.

The difference between Series II BF and Series II BFR in the average shearing stresses can be attributed to the presence of the web reinforcement.

The effect of the shear span-to-depth ratio is most apparent in the mode of failure. As the a/d increases the type of failure changes from diagonal tension to fatigue of the reinforcement which indicates the trend toward flexural failure.

Percentage of Web Reinforcement

The most apparent effect produced by the addition of stirrups was the change of the failure mode from diagonal tension to fatigue of the reinforcement. With the addition of stirrups the diagonal cracks penetrated deeper into the compression zone before the stirrups yielded. This is in agreement with the static tests of Wehr[20] and Harvey[9]. The presence of web reinforcement resulted in an increase of the fatigue life of the member. For example, Beam II BF-1 failed after 1500 cycles while its companion, Beam II BFR-1, had a life of 1,212,600 cycles.

Magnitude of Repeated Load

The magnitude of the repeated load had the expected effect on the fatigue life of the specimens. As the magnitude was increased, the life decreased. For some specimens the diagonal crack appeared during the static first cycle. With the exception of two specimens in Series I BF, the diagonal crack appeared after a few hundred cycles of loading for those specimens where it had not appeared on the first cycle.

ANALYSIS OF TEST DATA

Nominal Shearing Stress at Diagonal Cracking

The Joint ACI-ASCE Committee 326[2] presented a semi-empirical formula for predicting the resistance to diagonal tension cracking which was mentioned earlier. The equation read:

$$v_c = \frac{V_c}{bd} = 1.9 \sqrt{f'_c} + 2500 \frac{pVd}{M} \tag{3}$$

This equation is intended to predict the average shearing stress required to produce diagonal cracking at the section under consideration and is actually a lower limit when a/d is not too large. In the beams tested, the critical section was one effective depth (d) from the support. Hence, V/M = 1/(a-d) for the beams of this investigation. Using the above equation, the test results are compared in Table 5.

The prediction of Equation 3 was fairly accurate for most of the beams which were tested with repeated loads. For two beams where the equation was conservative the concrete strength was likely greater at the time of cracking.

Nominal Shearing Stress at Ultimate Load

A comparison of the measured values and the calculated values for ultimate shearing stress is given in Table 5.

Table 5. Comparison of Test Strengths with ACI-ASCE Committee Recommendations(2)

Beam	a/d	Diagonal Cracking Strength			Ultimate Shear Strength		
		v_c test (psi)	v_c calc.* (psi)	$\frac{v_c \text{ test}}{v_c \text{ calc.}}$	v_u test (psi)	v_u calc.* (psi)	$\frac{v_u \text{ test}}{v_u \text{ calc.}}$
I BF-1	2.16	285	173	1.65	297	173	1.72
I BF-2	2.16	156	158	.99	169	158	1.07
I BF-3	2.16	207	152	1.36	223	152	1.47
I BF-4	2.16	292	162	1.80	308	162	1.90
I BF-5	2.16	161	143	1.13	215	143	1.50
I BF-6	2.16	176	149	1.18	358	149	2.40
I BF-7	2.16	166	157	1.06	166	157	1.06
II BF-1	2.88	139	138	1.01	154	138	1.11
II BF-2	2.88	144	154	.95	144	154	.95
II BF-3	2.88	116	150	.77	204	150	1.36
III BF-1	3.96	116	145	.80	118	145	.82
III BF-2	3.96	98	141	.70	98	141	.70
III BF-3	3.96	137	151	.91	178	151	1.17
II BFR-1	2.88	163	152	1.06	193	240	.81
II BFR-2	2.88	163	152	1.06	165	240	.69
II BFR-3	2.88	149	157	.95	223	245	.92

* $v_c = V_c/bd = 1.9\sqrt{f'_c} + 2500\, pVd/M$. $v_c = v_u$ for beams without stirrups.

** $v_u = V_u/bd = v_c + Krf_{vy}$ for beams with stirrups.

For members with web reinforcement, the Joint Committee 326 recommended the following formula for ultimate shear strength:

$$v_u = \frac{V_u}{bd} = v_c + v_s$$

In this equation, v_s is the portion of the total shear assumed to be carried by the stirrups, as given by the truss analogy. Thus, $v_s = K(A_v/bs)f_{vy}$, or in this case where K = 1, $v_s = rf_{vy}$ (vertical stirrups). Therefore,

$$v_u = \frac{V_u}{bd} = 1.9\sqrt{f'_c} + 2500\frac{pVd}{M} + rf_{vy} \qquad (6)$$

This prediction gave reasonably good results for most of the specimens although it was unconservative when the failure mode was fatigue of the reinforcement as would be expected when a shear failure did not occur.

In Table 6 the test results are compared to the current design criteria of the "Standard Specifications for Highway Bridges"[4]. These specifications state that the allowable shearing stress (v_a) for beams without stirrups is 0.03 f'_c with a maximum limit of 90 psi. The shear stress is computed using v_c = V/bjd. For beams with stirrups, the allowable shear stress is given by

$$v_a = \frac{V}{bjd} = 90 + rf_v \qquad (7)$$

Table 6. Comparison of Test Strengths with AASHO "Standard Specifications for Highway Bridges"(4)

Beam	Percent of Ultimate Load Repeated	v_u test** (psi)	rf_v (psi)	v_a* (psi)	$\frac{v_u \text{ test}}{v_a}$
I BF-1	28.5	339	-	90	3.77
I BF-2	50	193	-	90	2.14
I BF-3	70	255	-	90	2.84
I BF-4	40	352	-	90	3.91
I BF-5	60	246	-	90	2.73
I BF-6	60	409	-	90	4.54
I BF-7	50	189	-	90	2.10
II BF-1	70	175	-	90	1.95
II BF-2	60	165	-	90	1.83
II BF-3	50	233	-	90	2.59
III BF-1	70	135	-	90	1.50
III BF-2	60	112	-	90	1.25
III BF-3	80	203	-	90	2.25
II BFR-1	70	221	43.9	134	1.65
II BFR-2	60	189	43.9	134	1.41
II BFR-3	80	255	43.9	134	1.91

* v_a = 90 psi for beams without stirrups.

$v_a = 90 + rf_v$ for beams with stirrups.

** v_a test $= v_u/bjd$ or practically $8\ v_u/7bd$.

In Equation 7, f_v is the working stress of the stirrup steel. The stirrup steel used in Series II BFR was a soft steel with a yield point of 23,000 psi; thus, a working stress of 11,500 was used.

The smaller shear span-to-depth ratio had the largest factor of safety as was found in static tests. The factor of safety decreased as the a/d ratio increased. The magnitude of the repeated load appeared to have a random effect on the factor of safety for the specimens of Series I BF. The results of the other series indicated that the safety factor decreased with decreasing magnitude of the repeated load.

Ultimate Strength in Flexure

The occurrence of fatigue failure of the reinforcement in several specimens suggest the influence of flexure. By Whitney's ultimate strength approach (f_y = 72,000 psi and f'_c = 5000 psi),

$$M_u = A_s\ f_y(d - a/2) \quad \text{where } a = \frac{A_s\ f_y}{.85\ f'_c b}$$

$$M_u = 626{,}000 \text{ in-lbs. (neglecting the compression steel)}$$

For ultimate crushing adjacent to the support block, this yields:

Beam	Calculated Flexure Moment	Calculated Shear Moment	Test Value
I BF-7	$P = \frac{626}{7.61} = 82.1$ k	P = 63.9 k	32 k
III BF-1	$P = \frac{626}{10.38} = 60.4$ k	P = 44.7 k	32 k
III BF-2	P = 60.4 k	P = 43.8 k	26.5 k
III BF-3	P = 60.4 k	P = 46.3 k	37 k
II BFR-1	$P = \frac{626}{9.32} = 67.2$ k	P = 59 k	41.5 k
II BFR-2	P = 67.2 k	P = 59 k	35.5 k
II BFR-3	P = 67.2 k	P = 60 k	48.0 k

As can be seen from the above comparison, the ultimate loads predicted by a flexural failure are not substantially greater than the ultimate loads predicted by shear for beams of Series III BF and Series II BFR. The effect of strain hardening has not been considered.

Moment at Shear-Compression Failure

Earlier studies[11,14,15,21] have shown that the load at shear failure may be correlated on the basis of the failure moment at the critical section. Morrow[15] and Moody[14] have each developed empirical expressions that predict the ultimate moment for sections weak in shear. Since the recommendations of Joint Committee 326 do not provide for additional strength beyond cracking of members without web reinforcement, the shear-moment expressions were used to predict

the ultimate strength of the members in this investigation. In Harvey's investigation[9] Morrow's expression was found to under-estimate the ultimate load while Moody's expression was conservative. An average of the estimates of Moody and Morrow was used as the basis for prediction of ultimate strength as it provided better results for the type of specimen being investigated. The two shear-compression expressions and sample calculations may be found in Appendix C.

Comparison of Static and Repeated Loadings

Series I BF beams are similar to Harvey's Beam IB-1[9]. The static failure mode was shear-compression, while most of the specimens tested with repeated loading failed by diagonal tension. Harvey's Beam IIB-1 also failed by shear-compression although companion beams of Series II BF failed by diagonal tension. Beams of Series II BFR, which were essentially the same as the statically tested Beam IIB-3, all failed by fatigue of the reinforcement while Beam IIB-3 failed by shear-compression. The same trend was evident for the beams of Series III BF when compared to the diagonal tension failure of Beam IIIB-1. Thus, the static failure mode may not necessarily be the same under repeated loads.

SUMMARY AND CONCLUSIONS

1. The beam tests reported herein indicate three general modes of failure in reinforced concrete beams without web reinforcement which are weak with respect to shear and are subjected to repeated loads:
   a. A "shear-compression" failure occurring at the section of maximum moment and at loads greater than the load at which the diagonal crack first penetrated the compression zone. Failure was by crushing of the reduced compression zone adjacent to the support.
   b. A "diagonal tension" failure occurring generally at some distance away from the support at a load equal to or slightly greater than the load at which the critical diagonal tension crack formed. Such failure was sudden, occurring with little warning.
   c. A "brittle fracture of the reinforcement" occurring suddenly and accompanied by the opening of an existing diagonal crack. This type of failure occurred at various load levels and after a considerable number of repetitions.
2. Beams with high percentages of stirrups cracked diagonally but failed by brittle fracture of the longitudinal reinforcement.

3. When the magnitude of the repeated load was reduced, the fatigue life of the member increased.
4. The semi-empirical formula for shear stress at diagonal cracking presented by the Joint ACI-ASCE Committee 326 was fairly reliable for members tested with repeated loads.
5. The factor of safety (ratio of ultimate shear stress to allowable shear stress) for beams, using the AASHO "Standard Specifications for Highway Bridges," (1965) ranged from 4.54 for the smaller a/d ratio down to 1.25 for a moderate a/d ratio for beams tested with repeated loads.
6. The shear stress at failure decreased with increasing a/d ratio under repeated loads just as has been found with static loads.
7. The presence of stirrups was found to greatly increase the resistance to failure under repeated loads.
8. For the limited number of specimens tested it was found that the failure mode for repeated loads did not coincide with the failure mode for static loads.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. ACI Committee 215, "Fatigue of Concrete," ACI Bibliography No. 3, ACI, 1960, 38 pp.

2. ACI-ASCE Committee 326, "Shear and Diagonal Tension," ACI Journal, Jan., Feb., Mar. 1962, Proceedings, Vol. 59.

3. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-63)," ACI Standard, June 1963.

4. American Association of State Highway Officials, "Standard Specifications for Highway Bridges," 1965.

5. Bower, J. E. and Viest, I. M., "Shear Strength of Restrained Concrete Beams without Web Reinforcement," ACI Journal, July 1960, Proceedings, Vol. 57, p. 73.

6. Chang, T. S. and Kesler, C. E., "Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement," ACI Journal, June 1958, Proceedings, Vol. 54, p. 1033.

7. Chang, T. S. and Kesler, C. E., "Fatigue Behavior of Reinforced Concrete Beams," ACI Journal, August 1958, Proceedings, Vol. 55, p. 245.

8. Elstner, R. C., Moody, K. G., Viest, I. M. and Hognestad, E., "Shear Strength of Reinforced Concrete Beams. Part 3 - Tests of Restrained Beams with Web Reinforcement," ACI Journal, Feb. 1955, Proceedings, Vol. 51, pp. 525-40.

9. Harvey, W. N., "A Study of Diagonal Tension Failure in Reinforced Concrete Beams," M.S. Thesis, Purdue University, 1964.

10. Hognestad, E., "What Do We Know About Diagonal Tension and Web Reinforcement in Concrete?" Circular Series No. 64, University of Illinois Engineering Experiment Station, March 1952.

11. Laupa, A., Siess, C. P. and Newmark, N. M., "Strength in Shear of Reinforced Concrete Beams," Bulletin No. 428, University of Illinois Engineering Experiment Station, 1955.

12. Lin, T. Y., "Strength of Continuous Prestressed Concrete Beams under Static and Repeated Loads," ACI Journal, June 1955, Proceedings, Vol. 51, p. 1037.

13. Magura, D. D. and Hognestad, E., "Tests of Partially Prestressed Concrete Girders," ASCE, Vol. 92, ST1, Feb. 1966, Paper 4685.

14. Moody, K. G. and Viest, I. M., "Shear Strength of Reinforced Concrete Beams - Part 4 - Analytical Studies," ACI Journal, March 1955, Proceedings, Vol. 51, p. 697.

15. Morrow, JoDean and Viest, I. M., "Shear Strength of Reinforced Concrete Frame Members without Web Reinforcement," ACI Journal, March 1957, Proceedings, Vol. 53, p. 833.

16. Nordby, G. M., "Fatigue of Concrete - A Review of Research," ACI Journal, August 1958, Proceedings, Vol. 55, p. 191.

17. Stelson, T. E. and Cernica, J. N., "Fatigue Properties of Concrete Beams," ACI Journal, August 1958, Proceedings, Vol. 55, p. 255.

18. Verna, J. R. and Stelson, T. E., "Failure of Small Reinforced Beams Under Repeated Loads," ACI Journal, October 1962, Proceedings, Vol. 59, p. 1489.

19. Verna, J. R. and Stelson, T. E., "Repeated Loading Effect on Ultimate Static Strength of Concrete Beams," ACI Journal, June 1963, Proceedings, Vol. 60, p. 743.

20. Wehr, K. E., "Shear Strength of Reinforced Concrete T-Beams," M.S. Thesis, Purdue University, 1967.

21. Zwoyer, E. M. and Siess, C. P., "Ultimate Strength in Shear of Simply-supported Prestressed Concrete Beams without Web Reinforcement," ACI Journal, October 1954, Proceedings, Vol. 51, p. 181.

APPENDIX A

APPENDIX A

Stress-Strain Properties of the Reinforcement

Several coupons of the steel reinforcement were selected and tested to determine the properties. The resultant stress-strain properties of an average test of the longitudinal steel are shown in Figure 43. The average stress-strain properties of the soft No. 4 wire are shown in Figure 44. The yield stress herein is defined as the upper yield point.

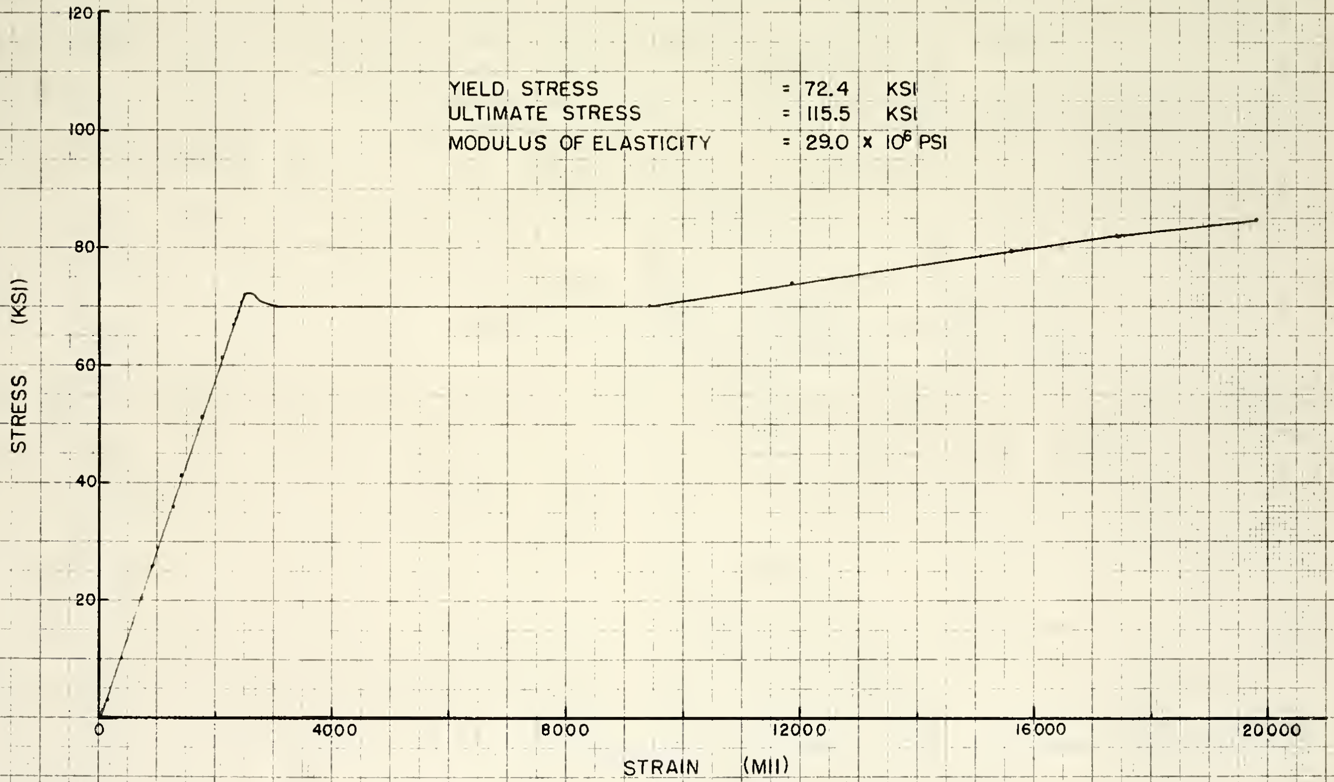


FIGURE 43. TYPICAL STRESS—STRAIN PROPERTIES — LONGITUDINAL STEEL

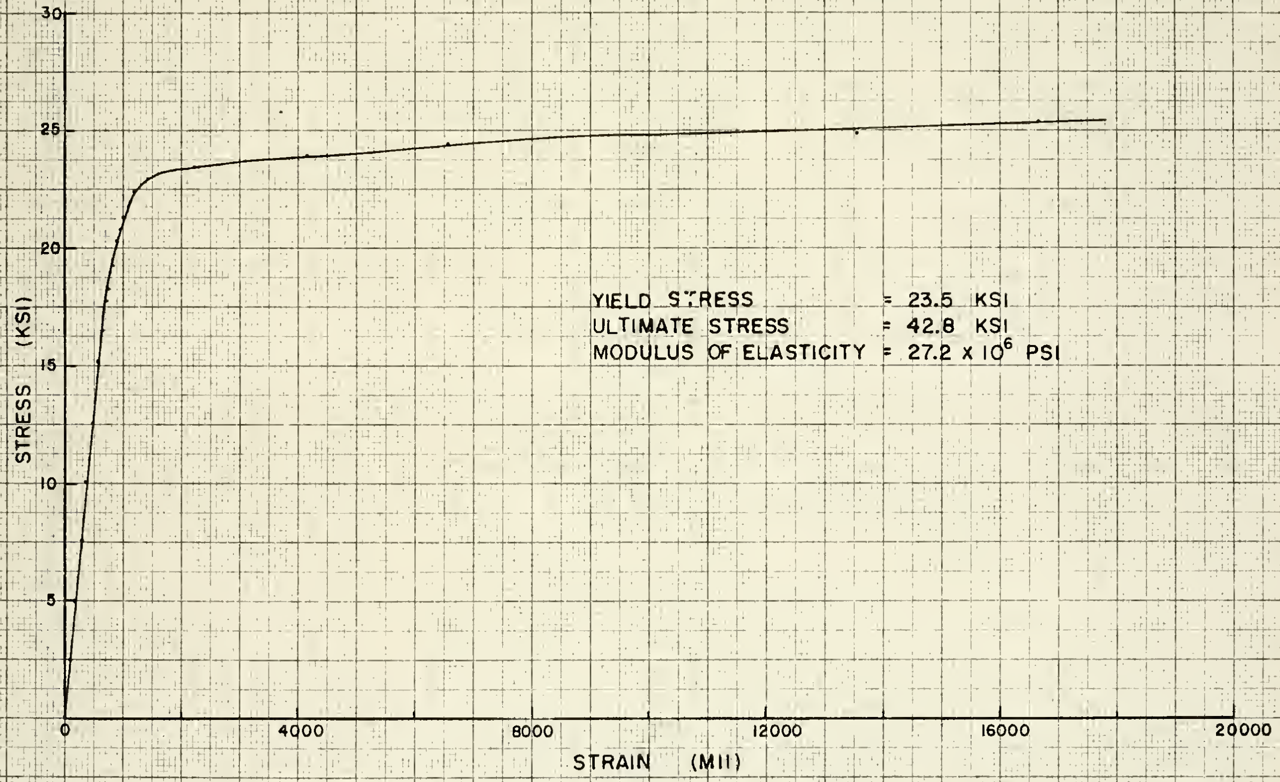


FIGURE 44. TYPICAL STRESS - STRAIN PROPERTIES — STIRRUP STEEL

APPENDIX B

APPENDIX B

Description of Loading Frame

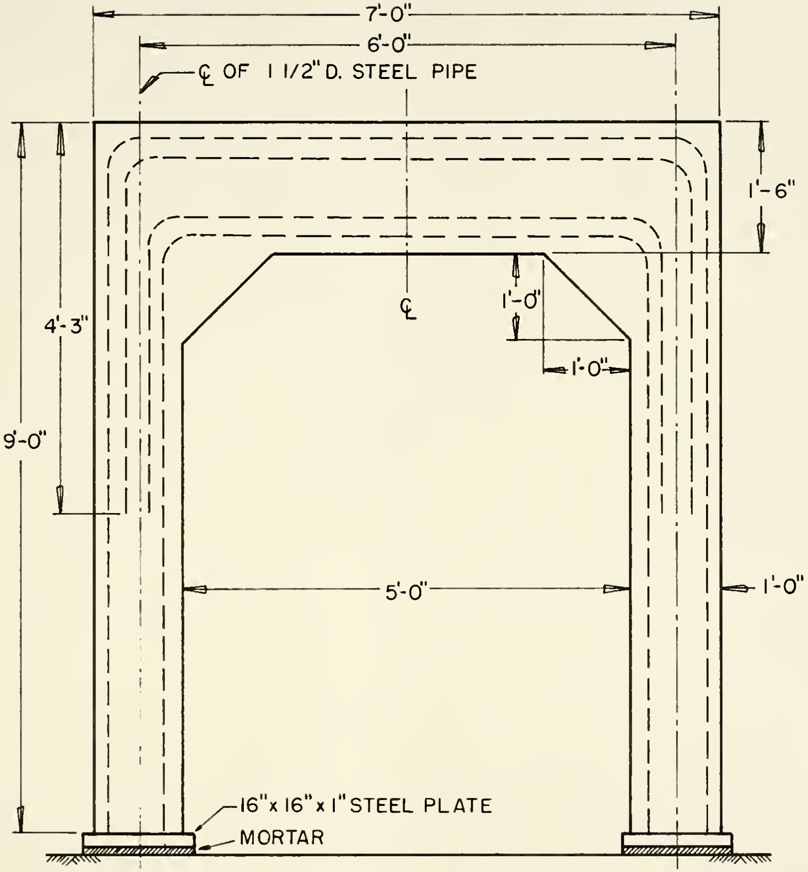
The first test of this project (I BF-1) was conducted with the Amsler hydraulic jack mounted on an existing steel frame in the laboratory. From observations during the first test it was apparent that the stability of the loading frame could be a problem in subsequent tests. As an alternative, a reinforced concrete frame was constructed for use during the remainder of the testing program.

The frame was designed to resist an upward load of 150 kips at the centerline of the frame using the working stress design criteria of the ACI Code. The resulting design was intended to be massive in order to improve stability characteristics. The steel reinforcement exceeded the requirements of ASTM A432 with an actual yield strength of 65 ksi. The concrete cylinder strength was 8310 psi at the age of 56 days although it had been assumed as 5000 psi for the design.

The frame was anchored to the laboratory floor with two Stressteel bars which were post-tensioned to a load of 50 kips each. The Stressteel bars were 1 1/8 inches in diameter and were threaded 3 inches on the ends to be received by plugs in the laboratory floor. The other ends were threaded with 8 inches of Acme thread to facilitate post-tensioning.

The yield strength of bars was assumed to be 70 per cent of the ultimate strength of 153,800 psi. Thus, the limiting load for each bar is 107.5 kips.

The details of the loading frame may be seen in Figures 45 and 46. The ultimate capacity of the frame for a load acting upward is limited by the capacity of the floor plugs, which is 100 kips for each plug. With an initial post-tension load of 50 kips on each bar, the maximum permissible load acting upward at the centerline of the frame is 100 kips. The ultimate flexural capacity of the horizontal cross-member is 515 ft.-kips which corresponds to a load at the frame centerline of 405 kips, and the ultimate shear capacity is 225 kips. The permissible load for the frame may be increased by decreasing the post-tensioning force, but will still be governed by the load limit of the floor plugs.



THICKNESS OF FRAME = 1'-1 3/4"

Scale: 1" = 20"

FIGURE 45. LOADING FRAME

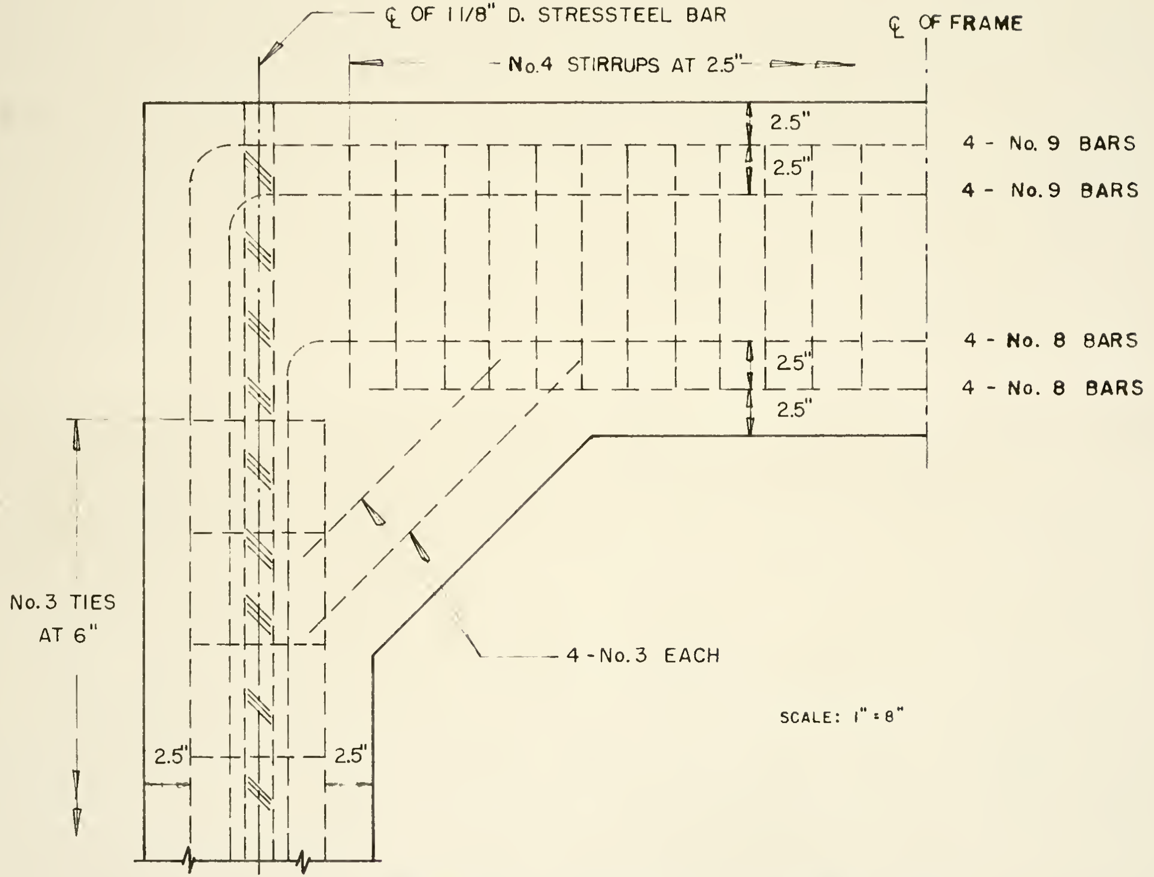


FIGURE 46. DETAILS OF LOADING FRAME

APPENDIX C

APPENDIX C

Sample Calculations for Ultimate Load Prediction

The ultimate load of a specimen without web reinforcement was estimated as the average of the values predicted by the shear-moment expressions of Moody[14] and Morrow[15]. For similar specimens of Harvey's investigation[9] this procedure seemed to be a reasonable estimate. The shear-moment criterion is very similar to that used for the ultimate strength in pure flexure. The effect of compression steel has been neglected.

Moody[14] developed the following expression for ultimate shear-moment for beams without stirrups:

$$M_s = pf_s\left(1 - \frac{k_2}{k_1k_3}\ \frac{pf_s}{f'_c}\right)bd^2$$ Eqn. (3a) Ref. 14

where $k_2 = .42$, $k_1k_3 = 1.121 - 0.0485\ \frac{f'_c}{1000}$ Eqns. (5a) and (5b) Ref. 14

And

$$f_s = \frac{3\ \frac{M}{Vd} - 0.45}{3\ \frac{M}{Vd} + 0.55}\left[6.9 \times 10^{-4}\ E_s\left(-1 + \sqrt{1 + \frac{1450}{pE_s/k_1k_3\ f'_c}}\right)\right]$$

Eqn. (6a) Ref. 14.

Morrow[15] developed the similar expressions:

$$M_s = p\, f_s\left(1 - \frac{k_2}{k_1 k_3}\ \frac{p\, f_s}{f'_c}\right)\ bd^2$$

$$\frac{k_2}{k_1 k_3} = 0.44, \qquad k_1 k_3 = \frac{800 + f'_c}{70 + f'_c}$$

$$f_s = \frac{1}{2}\ E_s\, K \epsilon_u \left(-1 + \sqrt{1 + \frac{4 k_1 k_3 f'_c}{p\, E_s\, K \epsilon_u}}\right)$$

$$10^4\ K \epsilon_u = \frac{1.116\ a/d + .174}{a/d - .872}$$

For beams with web reinforcement the recommendation of ACI-ASCE Committee 326[2] was increased by 12 per cent.

$$V_u = v_u bd = 1.9\ \sqrt{f'_c} + 2500\ \frac{p\ Vd}{M} + rf_v \qquad \text{(Eqn. 6)}$$

For all cases the calculated load was based on the moment, M_s, developed at the edge of the support block. (See Figure 4) Hence,

Series I: $$P_f = \frac{\text{Ave. } M_s}{7.61}$$

Series II: $$P_f = \frac{\text{Ave. } M_s}{9.32}$$

Series III: $$P_f = \frac{\text{Ave. } M_s}{10.38}$$

Sample Calculations

For Beam II BF-2:

$f'_c = 5580$ psi $\quad p = .01318$

$a/d = 2.88$

$E_s = 30 \times 10^6$ psi

Moody: $\quad M/Vd = \dfrac{9.32\ P}{.310\ P\ (11.13)} = 2.70$

$$f_s = \frac{3(2.7) - .45}{3(2.7) + .55} \left[6.9 \times 10^{-4}\ (30 \times 10^6) \right.$$

$$\left. \left(-1 + \sqrt{1 + \frac{1450}{(.0132)\ 30(10^6)\ /\ .85\ (5580)}}\ \right) \right]$$

$$f_s = .885 \left[2.029 \times 10^4\ (3.22) \right]$$

$f_s = 57{,}600$ psi $\quad k_1k_3 = 1.121 - .0485\ \dfrac{5580}{1000} = .850$

$$M_s = (.0132)\ (57{,}600)\ \left(1 - \frac{.42}{.850}\ \frac{.0132\ (57{,}600)}{5580}\right)\ 6(11.13)^2$$

$$M_s = 759\ (.933)\ (742.6)$$

$M_s = 525{,}000$ in.-lb.

Morrow: $\quad k_1k_3 = \dfrac{800 + 5580}{70 + 5580}$

$$10^4\ K\epsilon_u = \frac{1.116\ (2.88)\ +\ .174}{2.88\ -\ .872} = 1.69$$

$$f_s = 1/2\ (30 \times 10^6)\ (1.69 \times 10^{-4})$$

$$\left(-\ 1 + \sqrt{1 + \frac{4(1.13)\ 5580}{(.0132)(30 \times 10^6)(1.69 \times 10^{-4})}}\right)$$

$$f_s = 2510\ (-\ 1 + 19.55) = 46{,}500 \text{ psi}$$

$$M_s = (.0132)(46{,}500)(1\ -\ .44\ \frac{.0132\ (46{,}500)}{5580})(742.6)$$

$$M_s = 613\ (.952)(742.6) = 434{,}000 \text{ in.-lb.}$$

Average M_s = (525,000 + 434,000)/2 = 479,500 in.-lb.

$$P_f = \frac{479{,}500}{9.32} = 51.5 \text{ kips}$$

% of P_f = 60%

Maximum Applied Load = .6(51.5) = 31.0 kips.

For Beam II BFR-1:

f'_c = 5460 psi

p = .01318

r = .000381

$$M/Vd = 2.88$$

$$f_v = 23 \text{ ksi}$$

$$v_c = 1.9 \sqrt{5460} + 2500 \frac{(.0132)(11.13)(.310P)}{9.32P}$$

$$= 140.2 + 12.2 = 152.4 \text{ psi}$$

$$v_s = (.000381)(23{,}000) = 87.6 \text{ psi}$$

$$v_u = v_c + v_s = 240 \text{ psi}$$

$$V_u = v_u bd = 240\ (6)\ (11.13) = 16.1^k$$

$$P_f = V_u/.310 = 51.8^k$$

$$\text{Applied Load} = 1.12\ (.70)\ 51.8 = 41.5^k$$

APPENDIX D

APPENDIX D

Table D1. Steel and Concrete Strains - Beam I BF-1*

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar MII 3 1/2" From Support	No. 6 Bar MII 3 1/2" From Support	No. 6 Bar 16" From Support MII	No. 6 Bar 24" From Support MII	Compression Zone 3 1/2" from Support Bottom MII	Compression Zone 3 1/2" from Support 1" From Bottom MII
98,000	20.5	3.0	112 ± 200	358 ± 200	-33 ± 50	-13 ± 0	-14 ± 150	-103 ± 100
100,000	"	"	225 ± 200	386 ± 200	0 ± 10	-36 ± 10	-158 ± 100	-114 ± 100
150,000	"	"	---	404 ± 200	-18 ± 10	-36 ± 10	-158 ± 100	-116 ± 100
200,000	"	"	---	404 ± 200	0 ± 10	-25 ± 10	-158 ± 100	-116 ± 100
550,000	"	"	---	442 ± 200	20 ± 30	-10 ± 10	-154 ± 100	-116 ± 100
992,000	"	"	930 ± 200	400 ± 200	43 ± 30	-38 ± 10	-190 ± 100	-140 ± 100
1,239,200	"	"	1016 ± 200	432 ± 200	76 ± 30	-30 ± 10	-186 ± 100	-120 ± 100
1,629,500	"	"	1390 ± 200	710 ± 200	348 ± 50	---	---	---
1,800,000	"	"	1380 ± 200	765 ± 200	356 ± 50	---	---	---
2,176,300	"	"	1442 ± 200	766 ± 200	422 ± 50	---	---	---
2,508,900	"	"	1452 ± 200	752 ± 200	330 ± 50	---	---	---
3,033,200	"	"	1600 ± 200	898 ± 200	446 ± 50	---	---	---
4,043,300	"	"	1712 ± 200	962 ± 200	440 ± 50	---	---	---
5,256,600	"	"	1950 ± 200	1614 ± 200	512 ± 10	---	---	---
5,634,700	"	"	2046 ± 200	1918 ± 200	528 ± 10	---	---	---
5,947,100	"	"	2174 ± 200	2276 ± 200	466 ± 10	---	---	---
6,552,400	"	"	2382 ± 200	2860 ± 200	648 ± 10	---	---	---
7,098,890	"	"	2476 ± 200	3232 ± 200	664 ± 10	---	---	---
	Repeated Load Removed				-	-	-	-
	55		Diagonal Crack		-	-	-	-
	57		Failure					

* See Figure 16 for gage locations.

Table D2. Steel and Concrete Strains - Beam I BF-2*

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar 3 1/2"FS** South MII	No. 6 Bar 3 1/2" FS North MII	No. 6 Bar 16" FS North MII	No. 6 Bar 24" FS South MII	Compression Zone (All at 3 1/2" From Support) 2" S# MII	1" S MII	Bottom S MII	Bottom N## MII	1" N MII	[illegible] MII
800	13	3	138 ± 20	60 ± 20	36 ± 0	0	-85 ± 10	-145 ± 20	-110 ± 10	-72 ± 10	-86 ± 10	-50 ± 10
6,700	16	"	308 ± 40	150 ± 30	70 ± 0	0	-110 ± 10	-210 ± 20	-8 ± 10	-90 ± 20	-96 ± 10	-
17,300	32	"	630 ± 90	450 ± 100	800 ± 80	-	-	-	0 ± 10	-240 ± 40	-240 ± 40	-
18,900	32.5	"	630 ± 90	450 ± 100	860 ± 100	-	-	-860 ± 100	-	-	-245 ± 20	-
24,800	"	"	615 ± 80	480 ± 100	910 ± 100	-	-	-1000 ± 100	-	-	-300 ± 40	-
32,400	"	"	650 ± 100	440 ± 100	-	-	-	-1140 ± 100	-	-	-316 ± 40	-
72,500	"	"	-	680 ± 100	-	-	-	-1600 ± 120	-	-	-350 ± 50	-
98,500	"	"	-	570 ± 80	-	-	-	-1680 ± 130	-	-	-	-
132,000			Failure									

* See Figure 17 for gage locations.

** FS - from Support

\# South

\## North

Table D3. Steel and Concrete Strains - Beam I BF-3*

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar 3 1/2"FS** South MII	No. 6 Bar 3 1/2" FS North MII	No. 6 Bar 16" FS North MII	No. 6 Bar 24" FS South MII	Compression Zone (All at 3 1/2" From Support) (MII)					
							2" S#	1" S	Bottom S	Bottom N##	1" N	2" N
1	5	-	26	30	10	-12	-34	-26	-18	-36	-40	-32
1	10	-	90	144	28	- 7	-72	-54	-50	-90	-94	-78
1	15	-	246	430	36	-20	-104	-70	-90	-166	-172	-132
1	20	-	404	632	42	-18	-138	-90	-126	-230	-222	-160
1	25	-	560	755	53	-62	-176	-105	-176	-316	-295	-197
1	30	-	700	870	78	-28	-186	- 44	-268	-436	-392	-240
1	32	-	764	926	104	-32	-152	- 6	-291	-494	-440	-270
1	34	-	813	968	120	-74	-160	- 52	-312	-554	-490	-304
1	36	-	876	1022	156	-46	-182	-108	-310	-614	-536	-338
1	38	-	933	1079	208	-87	-210	-150	-317	-669	-568	-358
1	40	-	995	1134	259	-37	-232	-180	-325	-720	-603	-382
1	43	-	1096	1204	732	-30	-304	-318	-236	-841	-660	-379
2,200	43	3	608 ± 200	608 ± 200	1100 ± 300	700 ± 150	-310 ± 30	-200 ± 20	0 ± 100	-690 ± 200	500 ± 100	-380 ± 30
4,500	"	"	690 ± 200	760 ± 200	1210 ± 300	655 ± 200	-420 ± 150	-188 ± 50	-45 ± 200	-740 ± 250	-534 ± 250	-314 ± 120
6,500	"	"	700 ± 200	760 ± 200	1130 ± 300	634 ± 250	-500 ± 200	-188 ± 50	-620 ± 80	-740 ± 280	-620 ± 260	-340 ± 150
9,500	"	"	-	-	1132 ± 300	662 ± 240	-618 ± 200	-188 ± 50	-404 ± 280	-862 ± 260	-656 ± 260	-580 ± 180
12,600	"	"	-	-								
15,100			Failure									

* See Figure 18 for gage locations

** FS - From Support

\# S - South

\#\# N - North

Table D4. Steel and Concrete Strains - Beam I BF-4*

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar 3 1/2" FS** South MII	No. 6 Bar 3 1/2" FS North MII	No. 6 Bar 16" FS South MII	No. 6 Bar 24" FS North MII	Compression Zone All at 3 1/2" From Support: Bottom S# MII	Bottom N## MII	1"N MII	2" N MII
1	5	-	45	44	22	12	-24	-24	-29	-16
1	10	-	90	102	35	12	-75	-67	-75	-46
1	15	-	249	286	46	12	-143	-132	-142	-88
1	20	-	467	504	52	6	-214	-187	-198	-110
1	23	--	565	596	59	0	-268	-223	-230	-122
1	26.5	-	670	701	69	0	-322	-264	-264	-138
1,100	27	3	602 ± 260	584 ± 260	242 ± 60	24 ± 10	-286 ± 190	-180 ± 140	-162 ± 120	- 46 ± 50
45,000	"	"	652 ± 270	548 ± 270	353 ± 100	0 ± 10	-286 ± 180	-172 ± 180	-197 ± 120	-118 ± 60
77,500	"	"	682 ± 270	554 ± 270	353 ± 100	4 ± 10	-286 ± 180	-172 ± 180	-206 ± 120	-124 ± 60
250,600	"	"	732 ± 240	568 ± 260	394 ± 90	-12 ± 10	-286 ± 180	-178 ± 110	-224 ± 110	-138 ± 60
318,000	"	"	747 ± 260	575 ± 260	378 ± 100	-40 ± 10	-314 ± 160	-210 ± 110	-255 ± 110	-164 ± 60
418,000	"	"	786 ± 260	630 ± 250	444 ± 100	- 7 ± 10	-292 ± 170	-190 ± 110	-230 ± 110	-144 ± 60
630,000	"	"	754 ± 240	666 ± 240	417 ± 90	-57 ± 10	-330 ± 160	-232 ± 110	-274 ± 100	-184 ± 60
716,000	"	"	780 ± 260	650 ± 250	436 ± 100	-22 ± 10	-314 ± 170	-206 ± 120	-246 ± 110	-160 ± 60
830,600	"	"	740 ± 260	672 ± 250	426 ± 100	-36 ± 10	-316 ± 170	-204 ± 120	-252 ± 110	-170 ± 60
1,017,000	"	"	764 ± 250	748 ± 220	414 ± 90	-54 ± 10	-352 ± 160	-242 ± 110	-281 ± 110	-196 ± 60
1,095,400	"	"	780 ± 270	740 ± 260	464 ± 100	-10 ± 10	-308 ± 170	-186 ± 120	-240 ± 120	-150 ± 60
1,379,500	"	"	803 ± 240	---	404 ± 100	-32 ± 10	-326 ± 160	-202 ± 120	-260 ± 100	-186 ± 60
1,863,000	"	"	808 ± 240	---	456 ± 100	-45 ± 10	-348 ± 160	-224 ± 110	-278 ± 100	-215 ± 60
1,905,300	"	"	810 ± 240	---	456 ± 100	-50 ± 10	-356 ± 160	+236 ± 120	-294 ± 110	-215 ± 60
2,166,000	"	"	812 ± 240	---	456 ± 90	-48 ± 10	-360 ± 160	-242 ± 100	-295 ± 100	-218 ± 60
2,201,100	"	"	830 ± 240	---	478 ± 90	-24 ± 10	-347 ± 160	-226 ± 110	-284 ± 100	-206 ± 60
2,294,500	"	"	822 ± 250	---	489 ± 90	-14 ± 10	-335 ± 170	-206 ± 110	-265 ± 110	-188 ± 60

Table D4. Continued.

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar 3 1/2" FS** South MII	No. 6 Bar 3 1/2" FS North MII	No. 6 Bar 16" FS South MII	No. 6 Bar 24" FS North MII	Compression Zone All at 3 1/2" From Support: Bottom S# MII	Compression Zone All at 3 1/2" From Support: Bottom N## MII	Compression Zone All at 3 1/2" From Support: 1"N MII	Compression Zone All at 3 1/2" From Support: 2" N MII
2,536,600	27	3	820 ± 240	---	464 ± 80	-28 ± 10	-352 ± 170	-228 ± 110	-290 ± 110	-218 ± 60
2,636,600	"	"	850 ± 260	---	484 ± 100	+ 2 ± 10	-334 ± 170	-202 ± 110	-268 ± 110	-195 ± 60
3,023,000	"	"	858 ± 240	---	534 ± 100	0 ± 10	-322 ± 160	-192 ± 110	-262 ± 110	-176 ± 60
3,329,000	"	"	815 ± 240	---	498 ± 100	-44 ± 10	-351 ± 160	-226 ± 110	-308 ± 110	-230 ± 60

* See Figure 19 for gage locations.

** FS - From Support

\# S - South

\#\# N - North

Table D5. Steel and Concrete Strains - Be [illegible] F-5*

N(cycles)	Load (kips) Max.	Min.	No. 6 Bar 3 1/2" FS** North MII	Compression Zone(All 3 1/2" From Support) [illegible]tom North MII	1" North MII	2" North MII
1	5	-	32	-27	-29	-24
1	10	-	98	-67	-72	-58
1	15	-	257	-109	-114	-84
1	20	-	400	-190	-190	-134
1	25	-	512	-296	-270	-180
1	28	-	582	-348	-312	-199
1	31.3	-	675	-416	-366	-212
1,600	31.3	3	522 ± 270	-260 ± 170	-236 ± 160	-126 ± 80
5,000	"	"	563 ± 270	-230 ± 170	-208 ± 160	-106 ± 80
12,000	"	"	530 ± 260	-236 ± 140	-228 ± 120	-126 ± 70
22,800	"	"	500 ± 260	-226 ± 140	-224 ± 140	-122 ± 70
42,900	"	"	452 ± 260	-243 ± 150	-234 ± 140	-132 ± 80
58,600	"	"	470 ± 260	-224 ± 160	-222 ± 140	-127 ± 80
64,800	"	"	474 ± 260	-224 ± 160	-224 ± 140	-127 ± 80
111,800	"	"	480 ± 260	-224 ± 160	-224 ± 140	-126 ± 80
166,000	"	"	480 ± 270	-224 ± 160	-236 ± 140	-140 ± 80
333,900	"	"	492 ± 260	-227 ± 140	-238 ± 130	-130 ± 80
568,800	"	"	522 ± 270	-216 ± 160	-226 ± 140	-146 ± 80
752,900	"	"	527 ± 260	-211 ± 150	-233 ± 140	-134 ± 80
802,500	"	"	507 ± 260	-235 ± 150	-254 ± 140	-154 ± 80
815,500	41.5	3	614 ± 320	-266 ± 180	-273 ± 170	-160 ± 100
817,600	33.5	3	524 ± 240	-330 ± 190	-306 ± 150	-144 ± 60
820,000	31.5	3	524 ± 240	-328 ± 200	-312 ± 160	-152 ± 80

Table D5. Continued.

N(cycles)	Load (kips)		No. 6 Bar 3 1/2" FS**	Compression Zone (All 3 1/2" From Support)		
	Max.	Min.	North MII	Bottom North MII	1" North MII	2" North MII
829,000	34.3	3	551 ± 240	-[illegible] ± 220	-[illegible] ± 170	-162 ± [illegible]
839,000	34.3	3	554 ± 240	-[illegible] ± 220	-[illegible] ± 170	-170 ± [illegible]
868,300			Failure			

* See Figure 20 for gage locations.

** FS-From Support

Table D6. Steel and Concrete Strains - Beam I BF-6* (All strains measued at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone Bottom S** MII	Compression Zone Bottom N*** MII	Compression Zone 1" North MII	Compression Zone 2" North MII	Remarks
1	0	-	0	0	0	0	0	0	
1	5	-	25	28	-24	-22	-25	-22	
1	10	-	70	70	-63	-54	-59	-53	
1	15	-	392	417	-138	-114	-110	-90	
1	20	-	674	777	-217	-177	-158	-118	
1	25	-	784	904	-274	-230	-200	-144	
1	30	-	920	1056	-342	-275	-220	-138	
1	35.5	-	1084	1254	-518	-428	-276	-111	Diagonal crack at 34^K
1	35.5	-	1110	1280	-559	-467	-288	-106	
500	35.5	3	850 ± 420	837 ± 420	-352 ± 220	-278 ± 200	-167 ± 120	444 ± 150	
4,000	"	"	850 ± 400	837 ± 420	-268 ± 280	-197 ± 160	-204 ± 140	732 ± 200	
8,500	"	"	810 ± 480	756 ± 500	-258 ± 180	-167 ± 150	-221 ± 140	786 ± 180	
14,000	"	"	-	746 ± 500	-233 ± 160	-132 ± 150	-220 ± 140	-	
38,300	"	"	-	-	-212 ± 160	-120 ± 110	-222 ± 130	-	
802,000	"	"	-	-	-163 ± 160	-132 ± 80	-235 ± 140	-	Splitting along longitudinal steel.
915,200	"	"	-	-	-166 ± 160	-146 ± 80	-244 ± 130	-	
1,109,000	"	"	-	-	-192 ± 160	-178 ± 70	-278 ± 130	-	
1,273,700	"	"	-	-	-162 ± 170	-148 ± 60	-254 ± 140	-	
1,497,600	"	"	-	-	-182 ± 160	-166 ± 60	-280 ± 130	-	
1,647,200	"	"	-	-	-186 ± 160	-167 ± 60	-280 ± 140	-	
1,980,400	"	"	-	-	-160 ± 180	-166 ± 60	-270 ± 140	-	
2,385,000	"	"	-	-	-168 ± 180	-180 ± 60	-270 ± 140	-	Fatigue load terminated.
	55								Second diagonal crack forms.
	69.3								Failure

* See Figure 21 for gage locations.
** S - South
*** N - North

Table D7. Steel and Concrete Strains - Beam I BF-3* (All strains measured at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone Bottom S** MII	Bottom N*** MII	1" North MII	2" North MII	Remarks
1	5	-	29	32	-16	-29	-41	-27	
1	10	-	260	152	-51	-93	-100	-64	
1	15	-	545	473	-114	-186	-228	-142	
1	20	-	868	738	-166	-282	-315	-193	
1	25	-	1058	992	-211	-364	-392	-226	
1	30	-	1277	1062	-239	-441	-468	-259	
1	32	-	1360	1128	-248	-471	-495	-279	
1,000	32	3	971 ± 520	736 ± 440	-196 ± 140	-341 ± 240	-379 ± 240	-171 ± 140	Diagonal crack at 2000 cycles.
5,500	"	"	661 ± 520	735 ± 400	-119 ± 140	-406 ± 280	-440 ± 240	-192 ± 140	
10,000	"	"	591 ± 480	736 ± 400	- 90 ± 120	-428 ± 280	-453 ± 240	-199 ± 140	
58,000	"	"	-	756 ± 400	-	-364 ± 280	-411 ± 240	-210 ± 140	
108,500	"	"	-	770 ± 400	-	-360 ± 280	-432 ± 240	-258 ± 140	
297,500	"	"	-	920 ± 400	-	-353 ± 240	-446 ± 240	-306 ± 140	
366,700	"	"	-	-	-	-350 ± 240	-442 ± 240	-306 ± 140	
459,300	"	"	-	-	-	-330 ± 240	-432 ± 240	-323 ± 140	
700,000	"	"	-	-	-	-356 ± 240	-454 ± 240	-358 ± 140	
1,020,000	"	"	-	-	-	-331 ± 240	-461 ± 240	-400 ± 140	
1,152,500	"	"	-	-	-	-331 ± 240	-460 ± 240	-376 ± 140	
1,210,500	"	"	-	-	-	-330 ± 240	-482 ± 240	-372 ± 140	
1,448,500	"	"	-	-	-	-336 ± 240	-484 ± 240	-413 ± 140	
1,542,500	"	"	-	-	-	-318 ± 240	-473 ± 240	-378 ± 140	
1,615,700	"	"	-	-	-	-314 ± 240	-456 ± 240	-388 ± 140	
1,844,500	"	"	-	-	-	-306 ± 240	-449 ± 240	-393 ± 140	
2,000,000	"	"	-	-	-	-286 ± 240	-449 ± 240	-384 ± 140	
2,253,000	"	"	-	-	-	-338 ± 240	-484 ± 240	-435 ± 140	
2,641,000	"	"	-	-	-	-507 ± 240	-681 ± 240	-522 ± 120	Splitting along longitudinal steel.
2,800,000	"	"	-	-	-	-516 ± 240	-684 ± 240	-528 ± 120	
2,938,000	"	"	Failure						

* See Figure 22 for gage locations.

** S - South

*** N - North

Table D8. Steel and Concrete Strains - Beam II BF-1* (All gages at 3 1/2" from support.)

N (Cycles)	Load (kips)		No. 6 Bar North MII	Compression Zone				Remarks
	Max.	Min.		Bottom South MII	Bottom North MII	1" North MII	2" North MII	
1	5	-	48	-41	-35	-39	-28	
1	10	-	194	-118	-100	-113	-74	
1	15	-	506	-199	-168	-181	-110	
1	20	-	780	-287	-250	-250	-138	
1	25	-	1000	-380	-340	-324	-169	
1	30	-	1174	-724	-556	-298	+2288	Diagonal crack forms
1	33	-	1275	-788	-460	-464	+5840	
700	33.0	3	680 ± 600	-	-	-	-	
1,500	Failure							

* See Figure 26 for gage locations.

Table D9. Steel and Concrete Strains - Beam II BF-2* (All gages at 3 1/2" from support.)

N (Cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar North MII	Compression Zone 2" South MII	1" South MII	Bottom S** MII	1" North MII	Bottom N*** MII	2" North MII	Remarks
1	5	5	34	-30	-36	-39	-38	-54	-34	
1	10	-	89	-70	-74	-94	-94	-137	-88	
1	15	-	178	-124	-129	-182	-160	-244	-134	
1	20	-	316	-164	-169	-268	-224	-347	-174	
1	25	-	590	-194	-220	-372	-274	-455	-193	
1	31	-	924	-230	-288	-515	-338	-610	-210	
900	31	3	596 ± 340	-116 ± 80	-156 ± 120	-340 ± 220	-194 ± 140	-407 ± 250	-100 ± 80	Diagonal crack at 2200 cycles
4,700	"	"	570 ± 340	-116 ± 80	-170 ± 120	-378 ± 220	-136 ± 120	-453 ± 270	+218 ± 30	
7,600	"	"	570 ± 340	-116 ± 80	-204 ± 140	-436 ± 250	-130 ± 120	-466 ± 300	+360 ± 30	
8,800			Failure							

* See Figure 27 for gage locations.

** S - South

*** N - North

Table D10. Steel and Concrete Strains - Beam II BF-3* (All gages at 3 1/2" from support.)

N(cycles)	Load (kips)		No. 6 Bar South	No. 6 Bar North	Compression Zone				Remarks
	Max.	Min.	MII	MII	Bottom S** MII	Bottom N*** MII	1" North MII	2" North MII	
1	0	-	0	0	0	0	0		
1	5	-	39	38	-49	-62	-35	-25	
1	10	-	275	[illegible]	-143	-190	-80	-50	
1	15	-	660	660	-246	-330	-150	-42	
1	20	-	910	910	-334	-434	-138	-32	
1	25	-	1120	1110	-420	-540	-252	-110	
500	25.0	3	926 ± 500	794 ± 500	-296 ± 180	-392 ± 200	-150 ± 110	-40 ± 50	[illegible] beginning to occur
5,000	"	"	910 ± 500	812 ± 500	-338 ± 180	-463 ± 220	-160 ± 110	-30 ± 50	
10,400	"	"	810 ± 500	792 ± 500	-376 ± 180	-503 ± 220	-166 ± 110	-30 ± 50	
23,500	"	"	910 ± 500	930 ± 500	-426 ± 200	-572 ± 240	-170 ± 110	-42 ± 40	
60,000	"	"	914 ± 500	936 ± 500	-494 ± 200	-656 ± 250	-188 ± 110	-70 ± 30	
340,500	"	"	940 ± 500	-	-612 ± 220	-820 ± 260	-244 ± 120	-110 ± 30	
403,200	"	"	912 ± 420	-	-620 ± 220	-816 ± 270	-248 ± 120	-110 ± 30	
463,000	"	"	932 ± 400	-	-610 ± 220	-816 ± 270	-240 ± 120	- 94 ± 40	
680,000	"	"	-	-	-632 ± 220	-816 ± 270	-268 ± 120	-116 ± 40	
743,000	"	"	-	-	-636 ± 200	-808 ± 270	-268 ± 120	-116 ± 40	
875,000	"	"	-	-	-662 ± 220	-834 ± 270	-293 ± 120	-135 ± 40	
1,069,000	"	"	-	-	-650 ± 220	-834 ± 270	-302 ± 120	-135 ± 40	
1,152,800	"	"	-	-	-634 ± 220	-808 ± 270	-285 ± 120	-120 ± 40	
1,267,700	"	"	-	-	-642 ± 220	-816 ± 270	-302 ± 120	-135 ± 40	
1,464,400	"	"	-	-	-622 ± 220	-792 ± 260	-300 ± 120	-118 ± 40	
1,835,500	"	"	-	-	-588 ± 220	-732 ± 270	-340 ± 120	-125 ± 40	
1,918,500	"	"	-	-	-546 ± 200	-686 ± 260	-344 ± 120	-110 ± 40	
2,029,400	"	"	-	-	-532 ± 200	-650 ± 270	-360 ± 120	-106 ± 40	
2,229,500	"	"	-	-	-510 ± 200	-629 ± 270	-388 ± 120	-116 ± 40	
2,316,?00	"	"	-	-	-488 ± 200	-604 ± 270	-370 ± 120	- 90 ± 40	

Table D10. Continued.

N(cycles)	Load (kips)		No. 6 Bar South	No. 6 Bar North	Compression Zone				Remarks
	Max.	Min.	MII	MII	Bottom S** MII	Bottom N*** MII	1" North MII	2" North MII	
2,397,500	25.0	3	-	-	-498 ± 200	-603 ± 270	-386 ± 120	-108 ± 40	
2,596,100	"	"	-	-	-498 ± 200	-607 ± 260	-420 ± 140	-132 ± 40	
2,761,500	"	"	-	-	-486 ± 200	-590 ± 270	-412 ± 140	-116 ± 40	
2,983,000	"	"	-	-	-492 ± 180	-602 ± 250	-438 ± 130	-134 ± 40	
3,000,000	"	"			-486 ± 180	-592 ± 250	-436 ± 130	-128 ± 40	
									Repeated load terminated
	44		Failure						

* See Figure 28 for gage locations.

** S - South

*** N - North

Table D11. Steel and Concrete Strains - Beam III BF-1* (All strains measured at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone Bottom S** MII	Compression Zone Bottom N*** MII	Compression Zone 1" North MII	Compression Zone 2" North MII	Remarks
1	0	-	0	0	0	0	0	0	
1	5	-	70	82	-66	-52	-55	-49	
1	10	-	342	319	-189	-142	-148	-127	
1	15	-	660	640	-334	-247	-254	-196	
1	20	-	966	927	-481	-352	-348	-250	
1	25	-	1230	1163	-620	-460	-448	-310	
1	31.5	-	1498	1402	-793	-580	-555	-374	
1,500	31.5	3	1040 ± 600	940 ± 560	-555 ± 380	-360 ± 260	-367 ± 240	-204 ± 180	Diagonal crack forms
9,500	32.0	"	1054 ± 600	-	-592 ± 380	-380 ± 260	-398 ± 240	-206 ± 180	
61,500	"	"	1144 ± 520	-	-610 ± 380	-352 ± 260	-405 ± 240	-204 ± 180	
113,500	"	"	1246 ± 400	-	-646 ± 380	-375 ± 240	-426 ± 240	-225 ± 180	
303,600	"	"	-	-	-648 ± 360	-358 ± 240	-408 ± 240	-210 ± 180	
419,000	"	"	-	-	-642 ± 400	-340 ± 240	-398 ± 240	-195 ± 180	
479,200	"	"	-	-	-660 ± 400	[illegible] ± 240	-427 ± 240	-219 ± 180	
1,057,200	"	"	-	-	-708 ± 400	[illegible] ± 260	-446 ± 240	-242 ± 180	
1,152,500	"	"	-	-	-672 ± 400	[illegible] ± 260	-420 ± 240	-210 ± 180	
1,290,600	"	"	Failure						

* See Figure 32 for gage locations.

** S - South

*** N - North

Table D12. Steel and Concrete Strains - Beam III BF-2* (All gages at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone: Bottom N** MII	Compression Zone: Bottom S*** MII	Compression Zone: 2" South MII	Compression Zone: 1" South
1	5	-	54	50	-49	-52	-48	-50
1	10	-	123	106	-120	-122	-108	-114
1	15	-	240	264	-219	-223	-182	-208
1	20	-	544	[illegible]	-330	-336	-255	-314
1	26.5	-	843	[illegible]	-488	-485	-305	-427
600	26.5	3	-	642 ± 320	-340 ± 210	-338 ± 210	-170 ± 120	-268 ± 200
3,000	"	"	-	660 ± 320	-378 ± 240	-380 ± 240	-196 ± 120	-298 ± 200
7,700	"	"	-	710 ± 320	-390 ± 240	-387 ± 220	-198 ± 120	-300 ± 200
92,400	"	"	-	-	-416 ± 200	-416 ± 220	-205 ± 120	-326 ± 200
290,000	"	"	-	836 ± 330	-476 ± 240	-472 ± 240	-250 ± 120	-352 ± 200
339,000	"	"	-	866 ± 330	-472 ± 240	-472 ± 240	-250 ± 120	-374 ± 200
517,000	"	"	-	1074 ± 280	-472 ± 240	-472 ± 240	-242 ± 120	-360 ± 200
682,000	"	"	-	1220 ± 280	-486 ± 240	-486 ± 240	-252 ± 120	-382 ± 200
889,400	"	"	-	1490 ± 340	-488 ± 240	-496 ± 240	-240 ± 120	-382 ± 200
1,092,500	"	"	-	1764 ± 340	-510 ± 240	-486 ± 240	-247 ± 120	-382 ± 200
1,184,700	"	"	-	1800 ± 340	-510 ± 240	-497 ± 240	-240 ± 120	-394 ± 200
1,267,800	"	"	-	1830 ± 340	-520 ± 240	-506 ± 240	-250 ± 120	-398 ± 200
1,444,000	"	"	-	2050 ± 340	-550 ± 240	-557 ± 240	-300 ± 120	-460 ± 200
1,540,600	"	"	-	2050 ± 340	-514 ± 240	-506 ± 240	-256 ± 120	-406 ± 200
1,839,600	"	"	-	2140 ± 340	-545 ± 240	-534 ± 240	-272 ± 120	-425 ± 200
1,907,700	"	"	-	2200 ± 340	-538 ± 240	-503 ± 240	-248 ± 120	-412 ± 200
2,015,500	"	"	-	2238 ± 340	-508 ± 240	-500 ± 240	-240 ± 120	-402 ± 200
2,210,000	"	"	-	2344 ± 340	-548 ± 240	-530 ± 240	-280 ± 120	-424 ± 200
2,301,000	"	"	-	2386 ± 340	-514 ± 240	-502 ± 240	-238 ± 80	-391 ± 200

Table D12. Continued.

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone Bottom N** MII	Compression Zone Bottom S*** MII	Compression Zone 2" South MII	Compression Zone 1" South MII
2,412,200	26.5	3	–	2400 ± 320	-516 ± 240	-507 ± 240	-244 ± 120	-415 ± 200
2,600,000	"	"	–	2530 ± 320	-568 ± 240	-552 ± 240	-280 ± 120	-455 ± 200
2,701,900	"	"	–	2548 ± 320	-510 ± 240	-492 ± 240	-226 ± 120	-392 ± 200
2,781,300	"	"	–	2530 ± 320	-522 ± 240	-510 ± 240	-243 ± 120	-416 ± 200
2,963,800	"	"	–	2632 ± 320	-540 ± 240	-520 ± 240	-260 ± 120	-428 ± 200
3,542,600	"	"	–	–	-540 ± 240	-525 ± 240	-246 ± 120	-413 ± 200
3,773,100	"	"	–	–	-750 ± 300	-522 ± 240	+560 ± 180	-318 ± 100
3,864,400			Failure					

* See Figure 33 for gage locations.

** N – North

*** S – South

Table D13. Steel and Concrete Strains - Beam III 8F-3* (All gages at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Min.	No. 6 Bar South MII	No. 6 Bar North MII	Compression Zone Bottom N** MII	Bottom S*** MII	1" South MII	2" South MII	Remarks
1	5	-	42	40	-34	-35	-33	-24	
1	10	-	108	112	-98	-97	-82	-69	
1	15	-	313	254	-195	-196	-145	-110	
1	20	-	548	476	-289	-285	-189	-126	
1	25	-	737	677	-386	-377	-234	-143	
1	30	-	952	934	-513	-490	-288	-160	
1	35	-	1138	1167	-651	-623	-345	-159	
1	37	-	1225	1270	-700	-676	-370	-153	
800	37	3	848 ± 520	782 ± 500	-598 ± 440	-400 ± 320	-506 ± 300	-	Diagonal crack forms
2,000	"	"	872 ± 520	838 ± 520	-490 ± 400	-408 ± 340	-493 ± 260	- 10	
5,000	"	"	872 ± 520	858 ± 520	-514 ± 400	-408 ± 340	-546 ± 260	0	
9,000	"	"	872 ± 520	883 ± 520	-490 ± 400	-357 ± 340	-603 ± 260	0	
43,500	"	"	890 ± 520	898 ± 520	-460 ± 400	-340 ± 300	-728 ± 320	0	
69,000	"	"	890 ± 520	920 ± 520	-474 ± 400	-370 ± 300	-760 ± 320	-	
88,000	"	"	Repeated load terminated because of instability						
	10	-	654	710	-188	-117	-402	- 52	
	20	-	978	1014	-545	-238	-766	- 90	
	30	-	1300	1300	-803	-351	-1284	-145	
	35	-	1450	1450	-916	-405	-1610	-163	
	37	-	1504	1506	-956	-387	-2120	-176	
	38	-	1558	1540	-960	-357	-2410	-184	
	39	-	1558	1596	-828	-406	-4150	-136	
	40	-	1588	1624	-843	-416	-4190	-136	Diagonal crack opens.
	41	-	1620	1662	-866	-424	-4280	-140	
	42	-	1650	1696	-882	-432	-4400	-144	
	43	-	1678	1728	-900	-439	-4504	-146	

Table D13. Continued.

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South	No. 6 Bar North	Compression Zone: Bottom N**	Compression Zone: Bottom S***	Compression Zone: 1" South	Compression Zone: 2" South	Remarks
			MII	MII	MII	MII	MII	MII	
	44	-	1718	1764	-916	-445	-4630	-152	
	45	-	1752	1792	-929	-448	-4778	-158	
	46	-	1787	1838	-940	-446	-4930	-164	
	47	-	1826	1877	-959	-424	-5183	-171	
	48	-	1854	1900	-	-372	-5640	-174	Failure

* See Figure 34 for gage locations.

** N - North

*** S - South

Table D14. Steel and Concrete Strains - Beam II BFR-1* (All gages at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Stirrup (a) MII	Stirrup (b) MII	Stirrup (c) MII	Compression Zone: Bottom S** MII	Compression Zone: Bottom N*** MII	Compression Zone: 1" North MII	Compression Zone: 2" North MII
1	5	-	47	54	0	0	0	-30	-30	-32	-2
1	10	-	170	[illegible]	5	0	0	-36	-36	-34	-30
1	15	-	370	380	-4	0	0	-174	-160	-147	-89
1	20	-	[illegible]	680	-3	0	0	-269	-252	-226	-120
1	25	-	880	890	0	0	-22	-372	-336	-262	-120
1	30	-	1165	1161	49	20	-19	-439	-430	-313	-119
1	35	-	1419	1412	143	124	-22	-620	-521	-370	-133#
1	41.5	-	1745	1759	619	476	0	-814	-714	-447	-174
2,000	41.5	3	1096 ± 600	1134 ± 620	344 ± 200	-	159 ± 80	-648 ± 400	-517 ± 320	-294 ± 200	-126 ± 100
10,000	"	"	1164 ± 640	1160 ± 640	603 ± 200	48 ± 400	267 ± 120	-668 ± 400	-474 ± 320	-370 ± 200	-133 ± 100
22,100	"	"	1160 ± 600	1156 ± 600	457 ± 280	224 ± 400	282 ± 120	-664 ± 400	-440 ± 320	-423 ± 240	-112 ± 100
49,500	"	"	1196 ± 600	1156 ± 600	1240 ± 200	-	356 ± 120	-637 ± 380	-458 ± 320	-496 ± 240	-108 ± 100
72,500	"	"	1205 ± 600	-	-	-	390 ± 120	-636 ± 380	-434 ± 320	-490 ± 240	-103 ± 100
101,000	"	"	1220 ± 600	-	-	-	435 ± 120	-636 ± 380	-428 ± 320	-490 ± 240	-106 ± 100
212,500	"	"	1330 ± 600	-	-	-	483 ± 120	-669 ± 380	-460 ± 320	-546 ± 240	-162 ± 100
365,500	"	"	-	-	-	-	560 ± 120	-712 ± 380	-491 ± 320	-610 ± 240	-134 ± 100
1,212,600			Failure								

* See Figure 38 for gage locations.

** S - South

*** N - North

\# Diagonal crack formed

Table D15. Steel and Concrete Strains - Beam II BFR - 2* (All gages at 3 1/2" from support.)

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Stirrup (a) MII	Stirrup (b) MII	Stirrup (c) MII	Compression Zone: Bottom S** MII	Compression Zone: Bottom N*** MII	Compression Zone: 1" North MII	Compression Zone: 2" North MII
1	5	-	57	45	5	5	-4	-35	-36	-36	-33
1	10	-	260	171	8	7	-4	-94	-100	-101	-90
1	15	-	722	658	0	6	-5	-221	-220	-197	-159
1	20	-	1016	925	0	12	-9	-310	-303	-259	-197
1	25	-	1240	1126	12	1	-14	-394	-380	-327	-230
1	30	-	1426	1300	69	-2	-16	-476	-462	-390	-269
1	35.5	-	1670	1529	255	18	-13	-582	-568	-471	-300#
1,000	35.5	3	995 ± 560	918 ± 520	280 ± 160	71 ± 20	-160 ± 10	-411 ± 240	-391 ± 240	-321 ± 200	-192 ± 120
10,000	"	"	998 ± 560	908 ± 520	336 ± 160	170 ± 60	-155 ± 0	-440 ± 240	-430 ± 240	-336 ± 200	-176 ± 120
24,500	"	"	1034 ± 540	-	374 ± 200	223 ± 80	-148 ± 0	-440 ± 240	-430 ± 240	-322 ± 200	-159 ± 120
44,000	"	"	-	-	278 ± 200	228 ± 80	-176 ± 0	-493 ± 240	-468 ± 240	-352 ± 200	-193 ± 120
87,000	"	"	-	-	-	272 ± 80	-129 ± 0	-477 ± 240	-470 ± 240	-359 ± 200	-170 ± 120
130,500	"	"	-	-	-	270 ± 80	-160 ± 0	-496 ± 240	-478 ± 240	-358 ± 200	-193 ± 120
305,800	"	"	-	-	-	318 ± 80	-298 ± 0	-502 ± 240	-500 ± 240	-370 ± 200	-200 ± 120
335,800	"	"	-	-	-	358 ± 80	-290 ± 0	-476 ± 240	-484 ± 240	-354 ± 200	-194 ± 120
371,500	"	"	-	-	-	374 ± 80	-270 ± 0	-476 ± 240	-490 ± 240	-355 ± 200	-197 ± 120
422,800	"	"	-	-	-	390 ± 100	-152 ± 0	-476 ± 240	-478 ± 240	-335 ± 200	-168 ± 120
502,500	"	"	-	-	-	396 ± 100	-166 ± 0	-479 ± 240	-496 ± 240	-358 ± 200	-190 ± 120
687,100	"	"	-	-	-	396 ± 100	-365 ± 0	-324 ± 240	-556 ± 240	-400 ± 200	-235 ± 120
729,200	"	"	-	-	-	464 ± 100	-296 ± 0	-191 ± 240	-497 ± 240	-359 ± 200	-192 ± 120
785,200	"	"	-	-	-	497 ± 100	-310 ± 0	-470 ± 240	-490 ± 240	-345 ± 200	-170 ± 120
930,000	"	"	-	-	-	543 ± 100	-274 ± 0	-486 ± 300	-505 ± 240	-	-
933,000	"	"	-	-	-	580 ± 160	-234 ± 10	-462 ± 280	-480 - 180	-326 ± 200	-150 ± 100
942,000	"	"	-	-	-	586 ± 160	-249 ± 10	-471 ± [illegible]	-471 ± [illegible]	-307 ± 200	-171 ± 100
1,314,000	"	"	-	-	-	890 ± 160	-	-550 ± 280	-550 ± 280	-383 ± 200	-66 ± 120
1,384,200	"	"	-	-	-	987 ± 200	-	-570 ± 280	-580 ± 280	-402 ± 200	-82 ± 120

Table D15. Continued.

N(cycles)	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Stirrup (a) MII	Stirrup (b) MII	Stirrup (c) MII	Compression Zone Bottom S** MII	Compression Zone Bottom N*** MII	Compression Zone 1" North MII	Compression Zone 2" North MII
1,570,400	35.5	3	–	-	–	1156 ± 200	–	-625 ± 280	-623 ± 280	-442 ± 200	- 73 ± 100
1,590,500	"	"	–	–	–	1190 ± 200	–	-592 ± 280	-592 ± 280	-410 ± 200	- 70 ± 100
1,625,500	"	"	–	–	–	1126 ± 200	–	-600 ± 280	-600 ± 280	-410 ± 200	- 62 ± 100
1,673,500	"	"	–	–	–	1397 ± 200	–	-600 ± 280	-614 ± 280	-430 ± 200	- 87 ± 100
1,708,300	"	"	–	–	–	1507 ± 200	–	-580 ± 280	-570 ± 280	-404 ± 200	- 48 ± 100
1,782,000	"	"	–	–	–	1620 ± 200	55	-590 ± 280	-584 ± 280	-393 ± 200	- 62 ± 100
1,846,400	"	"	Failure								

* See Figure 39 for gage locations.

** S – South

*** N – North

\# Diagonal crack at 35^k.

Table 26. Steel and Concrete Strains - Beam II BFR-3* (All gages at 3 1/2" from support.)

N cycles	Load (kips) Max.	Load (kips) Min.	No. 6 Bar South MII	No. 6 Bar North MII	Stirrup (a) MII	Stirrup (b) MII	Stirrup (c) MII	Compression Zone Bottom S** MII	Compression Zone Bottom N*** MII	Compression Zone 1" North MII	Compression Zone 2" North MII
1	5	-	42	42	0	0	18	-37	-33	-39	-39
1	10	-	335	313	0	0	33	-110	-103	-101	-100
1	15	-	742	693	0	12	36	-248	-221	-183	-160
1	20	-	978	900	0	7	49	-340	-289	-226	-187
1	25	-	1144	1096	140	0	58	-437	-360	-279	-214
1	30	-	1204	1317	270	0	66	-542	-435	-324	-240
1	35	-	1430	1488	351	84	79	-649	-522	-374	-255#
1	35'	-	1550	1533	376	157	250	-688	-547	-372	-244
1	40	-	1729	1712	428	288	110	-759	-646	-420	-261
1	45	-	1911	1906	613	499	124	-928	-796	-460	-300
1	49	-	2034	2046	630	624	146	-1043	-927	-452	-350
500	49.0	3	1000 ± 800	-	392 ± 400	494 ± 360	180 ± 20	-823 ± 480	-512 ± 400	-313 ± 240	-218 ± 120
4,500	"	"	-	-	476 ± 400	446 ± 360	-	-876 ± 480	-457 ± 400	-362 ± 240	-200 ± 120
10,000	"	"	-	-	-	510 ± 360	-	-926 ± 480	-434 ± 400	-380 ± 240	-180 ± 120
15,000	"	"	-	-	-	560 ± 360	-	-920 ± 490	-390 ± 400	-400 ± 240	-187 ± 200
20,000	"	"	-	-	-	584 ± 360	-	-920 ± 490	-390 ± 400	-406 ± 240	-154 ± 120
40,500	"	"	-	-	-	631 ± 360	-	-910 ± 480	-370 ± 400	-433 ± 240	-146 ± 160
100,000	"	"	-	-	-	718 ± 360	-	-906 ± 500	-335 ± 400	-462 ± 240	-140 ± 160
208,500	"	"	-	-	-	829 ± 360	-	-890 ± 500	-281 ± 400	-478 ± 240	-120 ± 160
296,400	"	"	Failure								

* See Figure 40 for gage locations.

** S - South

*** N - North

\# Diagonal crack at 37^k